MAY 27 1935

Engineering News-Record

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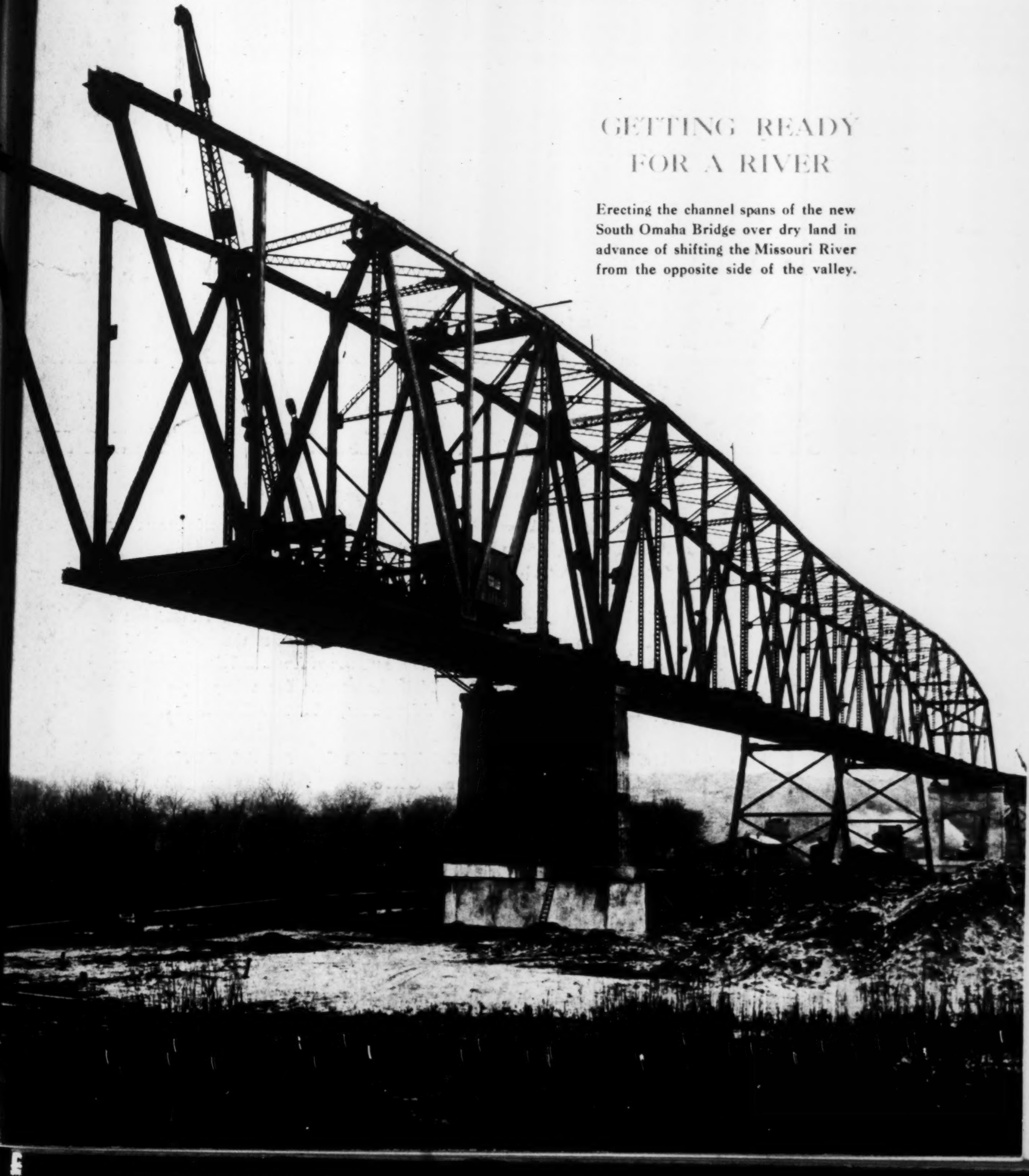
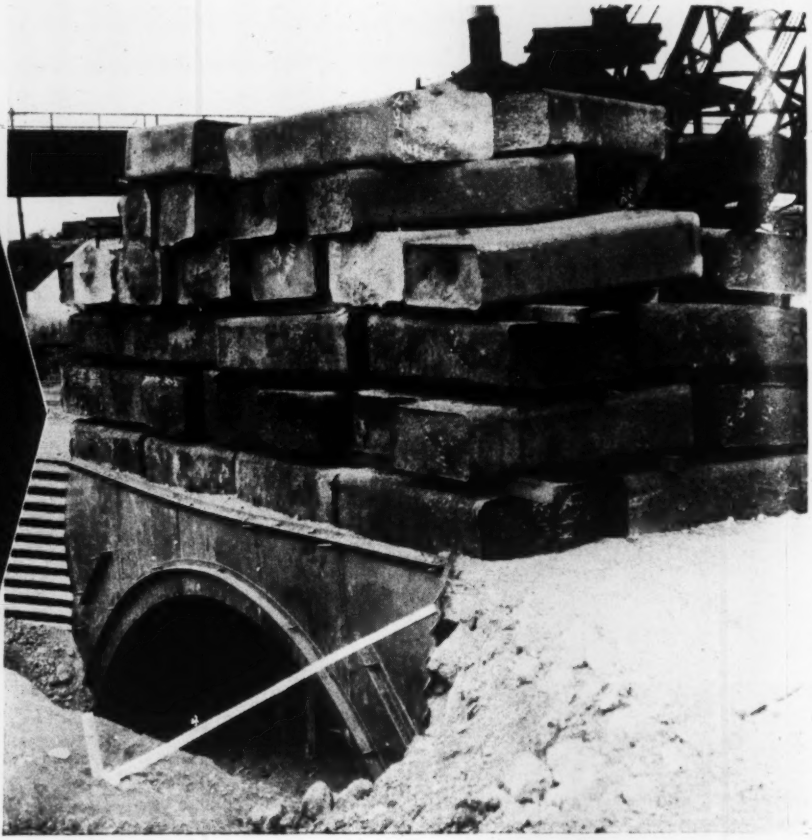


GETTING READY FOR A RIVER

Erecting the channel spans of the new South Omaha Bridge over dry land in advance of shifting the Missouri River from the opposite side of the valley.

350 TONS



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A MOMENT WITH THE PUBLISHERS

Time to Park the *Doctor's Kit*

LAST week this page dwelt on the fact of industrial recovery and asked business enterprise to shape its course accordingly, thereby fulfilling a fundamental responsibility to the people who, in the long run, can look only to business for their chance to earn a living.

On the shoulders of government, however, rests an equally fundamental responsibility; for with modern industry has come an increasing interdependence between government and business. This has not been a matter of conscious design or policy; it has been inevitable. So it is futile to sigh for a relationship that passed with the conditions of another century. The fact is that the two are inseparably entwined in the everyday life of every citizen.

IF democratic government is to survive the impact of twentieth century technical progress, business, as such, must exercise an increasing influence in the counsels of government. Henceforth, the welfare of the people will depend less and less on purely political issues and more and more on the practical and commercial applications of technical progress that we call industry and business. The stability of our employment and the abundance at our disposal will depend on how successfully government may foster productive industry and legitimate trade.

Equally is it true that government must exercise an increasing supervision over the affairs of business, especially of finance, so that a necessary concentration of economic power may not be perverted from creative and productive ends to pure acquisition and exploitation, thereby undermining the faith of the people in their political and economic institutions.

Business power that would frustrate the efforts of government to protect the people against exploitation and governmental power that would hamper or supersede the efforts of private enterprise to administer productively the economic life of the nation are equally dangerous to American institutions. Between these extremes we have no choice; either leads to disaster. Only through mutual respect and toleration can government and business best serve all the people.

Conscious of all this and in view of the conditions cited a week ago, this writer is persuaded that it is time for government to relax the hold it assumed two years ago in the exercise of its responsibility to this partnership. It may be true that business, in its panic, had sought security in a governmental protectorate, that in its distress, it had begged for the needle. It may have been willing even to trade its birthright of individual initiative and free competition for governmental protection against the hazards of the hour.

But that is no reason why government now should ignore the fact of convalescence. It is no reason why government should frighten the patient out of resuming his normal functions or plan to take over more of his activities. What though some business, in its hysteria, should have invoked a governmental straight-jacket to avert self-destruction? With restored mental balance we might now seek to relax such restraints rather than contemplate sewing them on permanently.

Even the most competent medical care seeks only to set up conditions under which nature can effect a cure. Sooner or later, if the patient is to recover, heroic and artificial measures alike must yield to the normal processes of convalescence. During the last two years sick business has had from its governmental doctors a variety of treatment—stimulants, restraints, correctives. Suppose, for a moment, that all these were necessary, well-chosen and beneficial. Assume, moreover, that a few corrective measures may still be in order to forestall a recurrence of the chronic ailment.

EVEN assuming all this, the time has come to park the doctor's kit and to give nature a chance. With respect to the future, it would be well to explain to the patient, clearly and definitely, just what should eventually be done and then to let his wounds heal up, his jaded nerves quiet down and his strength build up.

If it be the responsibility of business to carry on with confidence, it is equally the responsibility of government to encourage the process. And it is reasonable to surmise that unless the present doctors do let nature put the patient on his feet within the next twelve months, the American people will engage a new group of advisers. That's how the present staff got their jobs.

Willard Chevalier

ENGINEERING NEWS-RECORD

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VELOCITY CONTROL...

...Vital to the Engineer at His Throttle!

...All-Important to the Quarryman!

The right control of velocity—the right control of strength—gives the difference between day and night in quarry blasting. Rock conditions vary tremendously—hard, soft, brittle, tough, resilient, fine, coarse, just to mention a few of them—with every one, the quarryman's problem is to select the right explosive to give ideal control.

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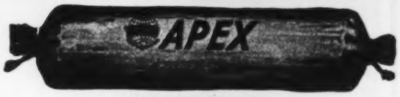

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EXPLOSIVES

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In the News:

The Big News of the Week concerned the relief-works program and originated in the Advisory Committee on Allotments, which made recommendations to the President for spending over a billion dollars. The Bureau of Public Roads was down for one-half of the total amount, the PWA Housing Division for one-quarter, the Corps of Engineers, the state of Wisconsin (under an interesting arrangement) and the Resettlement Administration for $100,000,000 each.

Big and Significant, too, was the announcement of wage rates that will apply on relief-works projects. They range, geographically, from $19 to $94 per month for 130 to 140 hours of work.

Fascinating Technically, but criticised on both economic and political grounds, the proposed Passamaquoddy tidal plant, near Eastport, Me., was recommended for an allotment of $10,000,000. The Corps of Engineers would build it.

Wisconsin lumped all of its non-federal projects together in applying for relief-works money. The 140 projects—city and state—aggregated some $200,000,000. An allotment of $100,000,000 was recommended, of which $70,000,000 would be a gift, subject to certain conditions.

Machinery was set in motion last week, by introduction of a bill by Senator Johnson of California, to remove the impasse on Parker Dam caused by the Supreme Court decision. Arizona wants its proposed Parker-Gila irrigation project in any compromise that may be worked out.

Denmark opened its $8,000,000 rail and highway bridge over the Little Belt last week, amid official ceremony. It joins one of the nation's principal islands to the mainland of Europe and thus makes Copenhagen (on another island) more easily accessible, by replacing ferries with new diesel streamlined trains.

Next Month has been set as the date for the opening of a new block of low-cost apartments by the Prudential Life Insurance Co. in Newark, N. J. This brings the company's investment to $2,330,000 for 734 apartments, which rent for $8.50 to $10.50 per room per month. The buildings cover 24 per cent of the plot, the interior of which is a municipally owned park.

A Bill, now in the final stages of enactment, will increase the bond-issuing power of the Home Owners Loan Corp. (distress mortgages) by $1,750,000,000, permit it to accept new loan applications, to invest $300,000,000 in private lending institutions, and to reduce premium rates and assessments for thrift institutions insured under the Federal Savings and Loan Insurance Corp. It is held to be a step in transferring home-mortgage lending activities from federal to private administration.

Passage of the Wagner Labor Disputes Bill by the Senate pleased union labor, since it establishes the "majority rule" for employee bargaining. The bill provides for the establishment of a "permanent labor relations board."

In This Issue:

A New Bridge is approaching completion over the Missouri River at Omaha. Some 4,300 ft. long, it includes a two-span, 1,050-ft., continuous truss for the channel crossing, which, in common with the rest of the bridge, was built on land in advance of a program of the Army Engineers to shift the channel from the east to the west side of the valley.

An Interesting Solution to a sewage-treatment problem, complicated by public policy as well as technical difficulties, is described in an article on the new plant at St. Albans, Vt. The outfall is mechanically ventilated for 6,000 ft. of its length, to serve as a supplemental aerator.

The Seventh Article in the series "Aerodynamics and the Civil Engineer" discusses the relation of wind velocity to height above the ground.

Oregon Highways now are designed for speeds up to 100 miles per hour, to make it possible to drive safely at normal speeds of 60 and 70 m.p.h. The high-speed design calls for new refinements in the layout of both horizontal and vertical curves.

Tunnel Driving and Lining at Fort Peck Dam constitute one of the major problems of that great undertaking. Early stages of the work are described in the second article reviewing the Fort Peck operations.

Discussion of the important question of load transfer at expansion and contraction joints in concrete pavements leads to a conclusion that effective shear joints are an economic necessity.

Coming Articles

Last Year when the fire that all but destroyed the Chicago stockyards leveled the famous old Amphitheatre, home of the International Live Stock Show, a decision was immediately made to build a new exhibition building, larger and more modern than its predecessor. The new building was in service within six months and in addition incorporated striking arrangements of both steel and concrete framing, which will be described in next week's issue.

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They cost less. They are more efficient. That is the simple success story of white concrete traffic markers.

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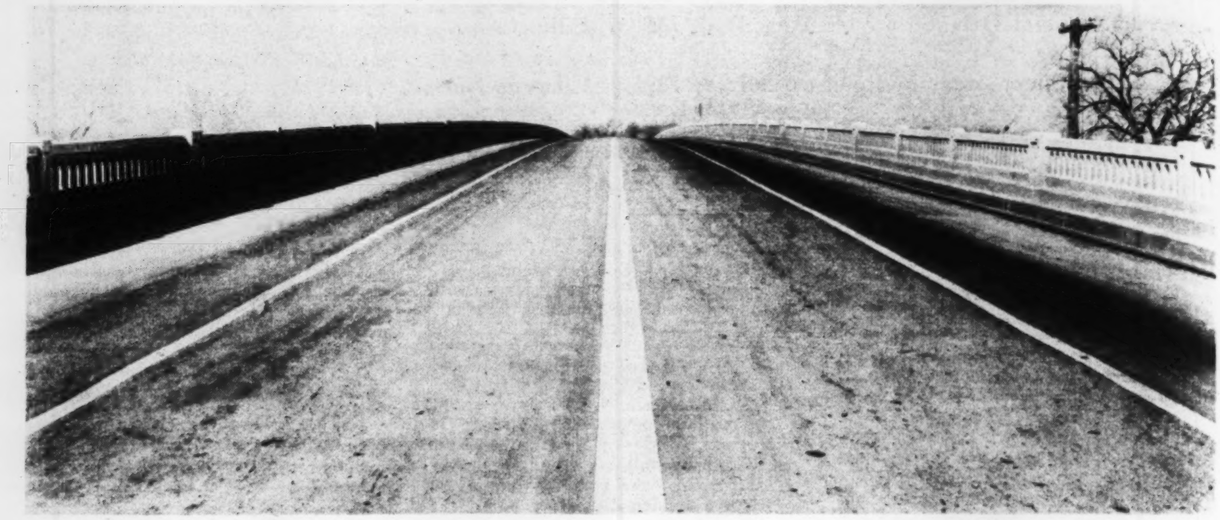

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May 23, 1935

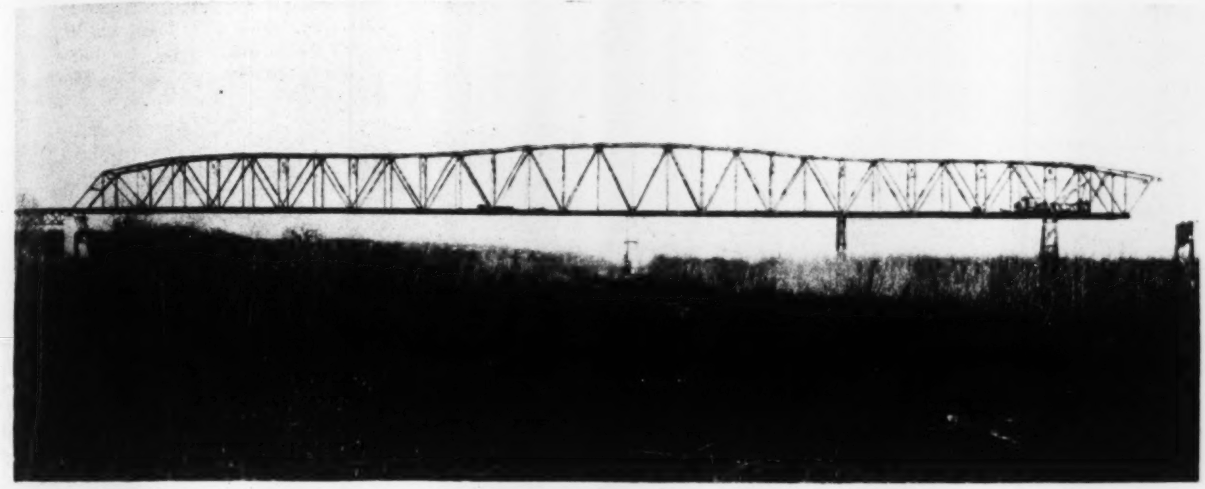


FIG. 1—FUTURE ROUTE of Missouri River barges. The continuous truss furnishes two 525-ft. navigation openings.

Building a Bridge for a Future River Channel

Construction of new Missouri River toll crossing at Omaha carried out on land by coordinating work with channel-shifting operations contemplated to place the channel under a 1,050-ft., two-span continuous truss in the mile-long structure

A NEW highway crossing of the Missouri River at Omaha, Neb., is rapidly approaching completion. Economically it is notable because it affords a direct connection between Iowa livestock-raising territory and the Union Stockyards in South Omaha, and because it permits traffic to bypass the congested streets of Council Bluffs, Ia., and the Omaha business district, which are connected by the only other vehicular river crossing in many miles—the Douglas St. Bridge owned by the street railway company. There are nineteen crossings of railroads at grade on the present route in Iowa; there will be at most one and probably none on the new route, which is also several miles shorter. Technically, the project commands interest by virtue of a 1,050-ft., two-span continuous truss founded on piers resting on rock 85 to 95 ft. below ground elevation, and because it contains nearly a mile of high-level viaduct approaches and a 5½-mile approach road built on 400,000 cu.yd. of earthfill. From a construction standpoint, the mile-long bridge is unusual if not unique, for practically all of it was built on land while the river channel was slowly being moved across the valley from east to west.

The South Omaha Bridge is being built by a special bridge commission of the city of Omaha as a toll project, financed by a $1,650,000 PWA loan and grant. A bridge at or near the present location was proposed about five years ago, but a bond issue of $2,000,000 to construct it was defeated in the 1930 elections. Put up again in 1931, the bond issue carried, and the engineers were employed. Failing to sell the bonds privately, the city formed the Omaha Bridge Commission in 1933 as a legal body to apply to the PWA for financing. The approved loan and 30 per cent grant were received in the fall of 1933. Self-liquidation will be effected through relatively low tolls corresponding to those now in force on the Douglas St. Bridge.

On their first design, the engineers had provided seven 400-ft. truss spans for the river crossing, supplemented by about 1,900 ft. of viaduct approach. The 2,800 lin.-ft. of through spans, all at a maximum height for navigation clearances, was required by the War Department because at the time no final plans had been made for the ultimate location of the river channel. It was deemed possible that a channel about 1,000 ft. wide might be established in any part of this width. About a year after the first War Department permit was granted, however, funds were made available for the improvement of the Missouri River, and the final location of the channel was fixed. The great number of long spans at high clearances then became unnecessary. A revised bridge plan was made with two 525-ft. navigation openings across the future location of the channel, and a new War

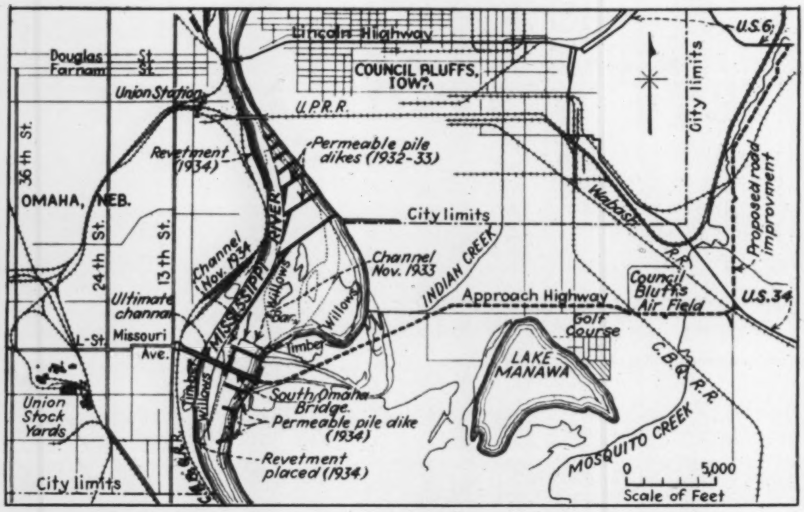

Douglas St.
Farnam St.
Union Station
Lincoln Highway
COUNCIL BLUFFS, IOWA
U.P.R.R.
36th St.
Revetment (1934)
Permeable pile dikes (1932-33)
City limits
OMAHA, NEB.
24th St.
13th St.
Channel Nov. 1934
MISSOURI RIVER
City limits
Channel Nov. 1933
INDIAN CREEK
Wabash
Proposed road improvement
Ultimate channel
Approach Highway
Council Bluffs Air Field
U.S.34
U.S.6
Willows
Bar
Timber
L-St.
Missouri Ave.
Golf Course
LAKE MANAWA
C.B.&Q.R.R.
Union Stock Yards
South Omaha Bridge
Permeable pile dike (1934)
Revetment placed (1934)
MOSQUITO CREEK
City limits
0
5,000
Scale of Feet

FIG. 2—LOCATION of new bridge and 5½-mile approach highway in relation to Omaha, Neb., and Council Bluffs, Ia. Present Iowa traffic bound for Omaha stockyards district or the West must cross the river at the business section of the city.

Department permit was granted for the revised plans. It was thus feasible to provide a descending grade to the east immediately beyond the east line of the channel, permitting the use of shorter and more economical deck spans and a considerable reduction in the total length of the bridge.

It was on these plans that the PWA financing was arranged. The change in plans effected a saving of some $400,000 over the originally planned expenditure, and approximately this amount of money is being spent by the War Department in directing the river channel under the two high-level navigation spans.

The contract for the bridge was let in the spring of 1934 for $1,035,468, and an additional contract involving $245,000 was awarded about the same time for grading work on the long Iowa approach road. The location of the bridge and of the former circuitous and heavily traveled route between U. S. Highway 6 or 34 in Iowa and the stockyards district in Omaha are shown on the map.

Design provisions

Fig. 3 is an elevation and plan of the bridge. The Omaha plaza at 13th St. and Missouri Ave. is on the western bluff of the river valley about 75 ft. above the bottom land. Beginning with two concrete girder spans, totaling about 107 ft., the western approach to the channel spans consists of 1,490 ft. of steel deck trusses on steel bents and towers, in spans varying in length from 159 to 111 ft. The channel crossing, a 1,050-ft. continuous truss in two spans of 525 ft. each, is followed by three 160-ft. deck truss spans, two 100-ft. girder spans and thirty 35-ft. steel beam spans on concrete piles that connect with the new approach road, a fill across river bottom, and farmland extending 5½ miles to a junction with U. S. Highway 34. In this approach road there are a plate-girder railroad grade-separation structure and a small concrete bridge over a creek. Exclusive of the approach fill, the bridge is 4,378 ft. long.

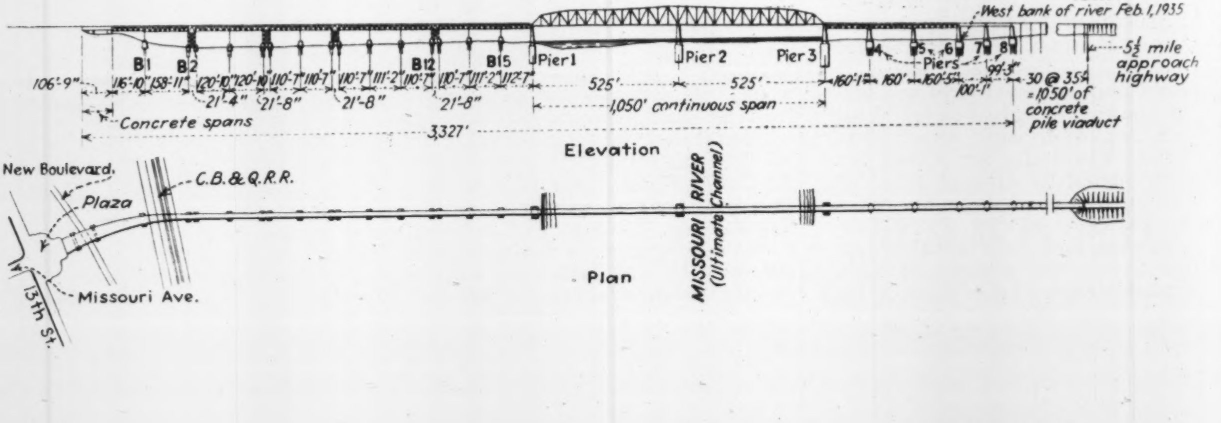
The bridge was designed for H-15 loading, but provision for design for double live load at 50 per cent increase over usual unit stresses gave a floor structure about equal to H-20 at ordinary unit stresses. The unit stresses given by the A.A.S.H.O. specifications were used throughout excepting only that an allowable unit stress in compression for double live load was utilized which conformed to the column formulas used in the new A.R.E.A. specifications for railway bridges, with due allowance for change in basic unit stress. For silicon steel, which was used in the continuous truss, the allowable unit stress in tension was taken at 36,000 lb. per sq.in. and in compression at $28{,}500-0.66L^2/R^2$. Throughout the design of the entire bridge, extensive use was made of 18-in. ship-channel sections, amounting almost to standardization.

Channel Spans—The 1,050-ft. span over the new channel of the Missouri River was constructed as a continuous span rather than as two simple spans both for economy and improved appearance. There was some saving in metal in the trusses, a saving in the number of floor joints, some reduction in the size of the center pier shaft due to only one shoe being required for the continuous span, and some saving in erection due to the elimination of temporary ties over the center pier. Floor panels are 32 ft. 9¾ in., and the floor system (28 ft. wide center to center of trusses) is arranged so as to use only three main stringers, 9½-ft. centers, with crossbeams on top of these stringers spaced at 3-ft. 3-in. centers, to carry the floor slab.

The 22½-ft. roadway slab is thickened ½ in. to allow for wear. Provision was made in the design for a future pavement weighing 20 lb. per sq.ft. An 18-in. wide curb composed of a steel channel laid flat is used on either side of the roadway. The slab reinforcing consists of pressure-welded trusses with temperature and distribution bars wired to the trusses at intersections. Except for the concrete pile trestle, all floor slabs have their main reinforcing parallel to the center line of the bridge.

Deck Trusses—The approach spans are of deck-truss construction, a Warren system being used throughout as

FIG. 3—PLAN and elevation of South Omaha Bridge project, which includes a 1,050-ft. continuous span, long deck truss and girder approaches and a 5½-mile approach highway.


IOWA
West bank of river Feb. 1, 1935
B1
B2
B12
B15
Pier 1
Pier 2
Pier 3
4
5
6
7
8
Piers
5½ mile approach highway
106'-9"
116'-10"
158'-11"
120'-10"
120'-10"
110'-7"
110'-7"
110'-7"
111'-2"
110'-7"
110'-7"
111'-2"
112'-7"
21'-4"
21'-8"
21'-8"
21'-8"
525'
525'
1,050' continuous span
160'-1"
160'
160'-5"
100'-1"
99'-3"
30 @ 35' = 1,050' of concrete pile viaduct
Concrete spans
3,327'
Elevation
New Boulevard
Plaza
C.B.&Q.R.R.
MISSOURI RIVER (Ultimate Channel)
Missouri Ave.
13th St.
Plan

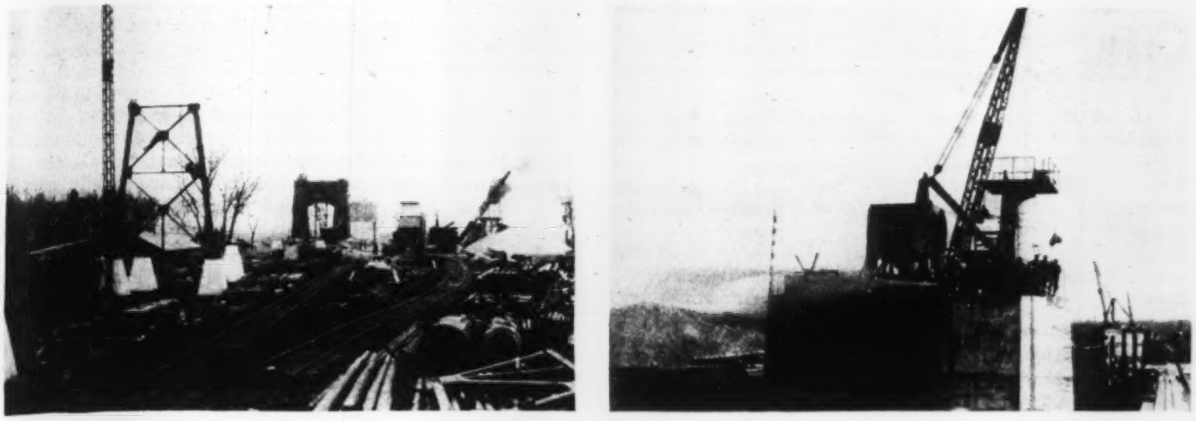


FIG. 4—CONCRETE for the piers was mixed at a central plant (left) located at about the center of the project. Transportation was by industrial railway in bottom-dump buckets that were handled into the forms by a crawler crane. Right: the last 20 to 40 ft. of sinking for the deep piers of the continuous truss span was done under air, with much of the material removed through blowpipes.

on the continuous span; the trusses are spaced 15½ ft. c. to c. The details on the approach trusses were reduced to a minimum by the use of rolled sections for all of the web system and built I-sections made of four angles and a web plate elsewhere. For the Nebraska approach viaduct, longitudinal rigidity is provided by towers spaced about 330 ft. c. to c., two steel bents being provided between each pair of towers. The truss spans are constructed as simple spans. At each bent a single shoe with two pins is provided on each column, with separate end posts and floor beams for each of the two spans meeting at the bent. The floor slab is discontinuous at this point, with the ends of the adjacent slabs protected by embedded angles.

Expansion joints on the approach viaduct are provided at the centers of the towers, each of the adjacent spans cantilevering out from the tower column to this point. The floor system is simplified to as great an extent as practicable by eliminating all stringers. The panel length of the trusses is in general 10 ft. with floor beams at each panel point; a 10-in. floor slab spans from floor beam to floor beam.

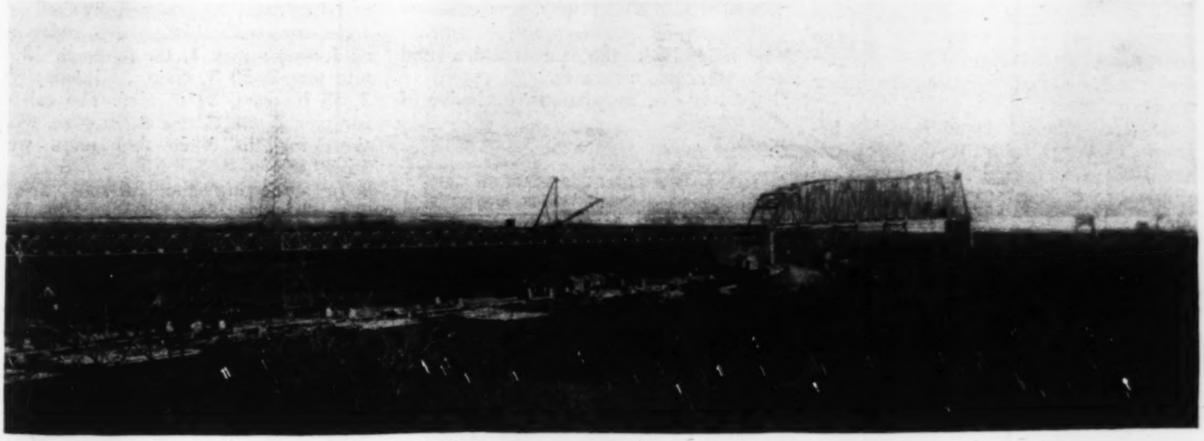
Deck Girders—On the concrete pile viaduct four stringers per span are used, spaced 6½ ft. on centers. The floor slab, 8½ in. thick, cantilevers beyond the outer stringers, to carry a concrete curb 18 in. wide. Expansion joints are provided at intervals of four spans, and the stringers are continuous for this length, being furnished in 70-ft. lengths and fully spliced at the joint between sections. At the expansion joints, which are located 3½ ft. from the center lines of the bents, a hanger detail is used to avoid a sliding joint for the beams.

Each bent consists of four precast piles 2 ft. square and 75 ft. long surmounted by a concrete cap. In order to simplify the bearing details of the stringers on the caps and to allow for errors in location of the pile bents, the sole plates are not bolted to the stringers but will be field-welded to the stringer flanges.

An inspection walkway is provided on all of the deck truss and deck girder spans at about the elevation of the bottom chord, to aid maintenance inspection.

Toll Plaza—Considerable attention is devoted to securing a good appearance at the Nebraska end of the structure where the bridge passes over a boulevard on an ornamental concrete span about 70 ft. long. The toll house is placed on a platform in the center of the roadway just beyond the west end of this concrete span, and canopies are provided over the roadways, to give protection to toll collectors. The roadway is widened at the toll house to give space for four lines of traffic, two on each side of the house. This necessary widening is effected by the use of long-radius curves connecting to the 22½-ft. roadway on the structure proper. A rather large plaza is provided between the toll house and 13th St., with lighting and traffic signals for control of traffic. Concrete or stone hand-rails will be provided along the edges of the plaza with appropriate planting outside. Concrete hand-rails will also be provided along the concrete span, but elsewhere throughout the structure the hand-rails will be of steel consisting of two 5-in. Z-bars connected by bar lacing.

FIG. 5—SOUTH OMAHA BRIDGE advancing east over dry land as Missouri River channel is being shifted west to a final position under the two through-truss spans. View shows steel-raising derrick on approach and central mixing plant near first "river" pier.



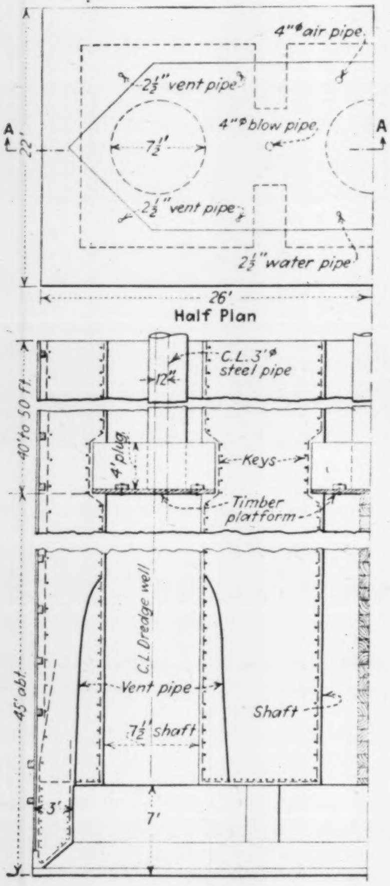


FIG. 6—CAISSON for deep piers. Sunk as far as possible by open-dredging through three 7½-ft. wells, the caisson was converted for air work by plugging two of the holes and installing a lock over the third.

The 5½ miles of approach connecting the east end of the bridge with the Iowa state highways consist largely of roadway on embankment. The highway is laid out with maximum curves of 2 deg. 30 min., with superelevation to care for a speed of about 50 miles per hour. A 20-ft. concrete pavement is provided, 7 in. thick in the center and 10 in. at the edges. It is reinforced throughout with wire mesh and is laid on subgrade paper. Joints filled with ¾-in. sponge rubber are provided at 40-ft. intervals; they are detailed for definite shear transmission.

The embankment is 34 ft. wide between shoulders, with 3-to-1 slopes, except that where guard fence is used the slopes are reduced to 1½ to 1. Steel band guard-rail is used where such protection is needed. The embankment is from 8 to 10 ft. high for length of somewhat over 2 miles adjacent to the river. Enough right-of-way was purchased to provide borrowpits throughout the length of the embankment. Right-of-way fencing is placed inside the borrowpit, to make these areas available for use by adjacent property owners.

Construction planning

In initiating the construction program, pier sinking was planned to coordinate with the river-regulation work of the army engineers, which involved shifting the channel from the east to the west side of the valley. By such coordination it was possible to construct all foundations on land. As indicated on the map (Fig. 2) a series of permeable pile spur dikes had been built in 1932 and 1933 upstream from the bridge site. These dikes, in eliminating a double channel that had existed just below the Union Pacific Bridge, stopped the destruction of valuable farmland on the Iowa side and shifted the channel about halfway to its final destination or until its west bank was just east of the location of pier 6 of the new bridge.

In 1934 dikes were installed below the new bridge site, and a further shift of the channel westward was begun. Fortunately for the bridge contractor, exceptionally low water reduced the bank-cutting power of the river, so that the shift was unusually slow. The plan followed was to build all of the foundations west of the channel as rapidly as possible, using land equipment, and to follow this by steel erection that would progress from west to east as the river channel was shifting from east to west. It was hoped by the time steel was up to about pier 6 (west bank of the channel at the start of the work), that the channel would have shifted far enough that the remaining piers 7 and 8 and the pile trestle could be built from land. If the channel did not move, these last foundations could be built with floating equipment. Such a program, involving postponing part of the foundations until after steel erection was well under way, was economically possible because a single contractor held both the substructure and superstructure contracts.

Twenty-five foundations, exclusive of the thirty pile bents, were required to be constructed. Three of these, piers 1, 2 and 3, which support the continuous truss, were to be carried to rock, which borings indicated was located at [illegible] —84, —85 and —94 ft. respectively. West of the continuous truss the foundations are shallow concrete pedestals on cast-in-place concrete piles, while east of this truss heavy footings on timber piles support tall concrete pier shafts.

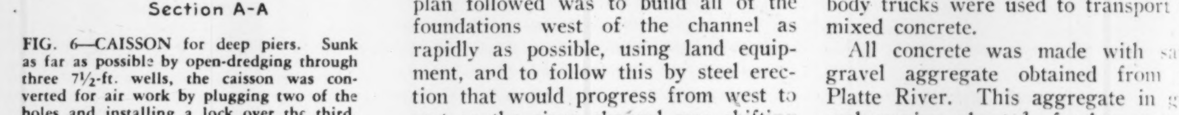
Foundation operations were begun at three points: pier 3, pier 4 and pier B15. Opposite the latter point a central concreting plant was installed and connected to the railroad running along the base of the Omaha bluff by a standard-gage track, placed parallel with the bridge center line. As shown in Fig. 4, the track bisected the plant layout, the aggregate stockpile and an elevated water tank being on one side and the bins, weighing batchers, cement house and mixer on the other. A locomotive crane was used to unload the material cars, to charge the bins from the stockpile and to raise platforms carrying cement sacks to the mixer floor. The 1-yd. mixer discharged on the opposite side of the plant. For the piers east of the plant, trains of three narrow-gage flat cars each carrying a 1-yd. bottom-dump bucket were run under the mixer discharge, and these cars were hauled to the pier sites by a gasoline locomotive operating over an industrial track laid on a pile trestle parallel with the bridge center line. For the pedestal foundations west of the plant, small tip-over-body trucks were used to transport the mixed concrete.

All concrete was made with sand-gravel aggregate obtained from the Platte River. This aggregate in general requires about ¾ of a bag more of cement than a stone aggregate, to secure equal strength. It was screened and recombined to secure a minimum fineness modulus of 3.85. The contracts were awarded on the basis of the contractor producing concrete to meet certain minimum required strengths for each class of work. All aggregates were weighed, and all concrete except that in the pier bases was vibrated.

The deep foundations for piers 1, 2 and 3 were started as open-dredged concrete caissons and completed under air, as follows: pier 1, 66 ft. open, 18 ft. air; pier 2, 50 ft. open, 35 ft. air; pier 3, 55 ft. open, 39 ft. air. The caisson for pier 2 was 22x52 ft. in plan, while those for the other two piers were 16x50 ft.

In designing the caissons special

FIG. 7—ERECTION DIAGRAM for 1,050-ft. continuous truss, showing location of steel erection bents.



care was taken to secure an arrangement that would have maximum efficiency for either open or pneumatic work. As a consequence, each caisson was supplied with three 7½-ft.-diameter dredging wells, to accommodate clamshell buckets easily. Then, for conversion to a pneumatic caisson, provision was made for installing plugs in each of the shafts and for connecting the airlock. As shown in Fig. 6, a keyway was left about 40 ft. above the bottom of each shaft. When air work was required, the concrete for the plug, which was about 4 ft. deep, was poured on a timber platform set in this keyway. A piece of 3-ft. pipe was placed in each plug, to serve as an opening for concreting the working chamber and the lower part of the shaft after the caisson had been sunk to rock and cleaned out. Also the air locks were attached to these pipes, the center lock being used as a manshaft and the two outside locks as material shafts.

Because most of the material encountered on these three piers was river mud and sand, only the bottom 2 ft. of the concrete cutting edge was armored with a steel plate; a steel angle with downstanding leg was used for the tip of the cutting edge. The caisson was equipped with blowpipes, and much of the material was removed through them (Fig. 4). Air was supplied by two diesel compressors located in a shed at the concrete mixing plant. An interesting fact in connection with these three piers was that rock was encountered at the exact elevation indicated by the wash borings.

For the five piers east of the main span, combination steel and wood cribs were used. After being sunk to required elevation, timber piles were driven under water and a tremie seal coat was placed. The remainder of the concrete for the footing was poured in the dry, following which the tall concrete pier shafts were constructed.

As stated, concrete was brought to the pier sites from the central plant in bottom-dump buckets on industrial-gage flat cars. For the foundations, these buckets were handled into the work by the stiff-leg derricks used for excavating, but for the tall pier shafts a crawler crane was utilized with a long boom operating on a corduroy road and timber pads.

Steel erection

Erection of the superstructure started at the Omaha end and progressed east, using a 15-ton steam traveler derrick on top in conjunction with a locomotive crane and a gin pole on the ground for the deck truss spans west of the through truss, thereafter using the traveler by itself. For the latter work, falsework was required; the majority of the deck trusses in the Omaha approach were raised from the ground complete without the necessity for falsework. Steel was brought in by rail, switched to the track alongside the bridge and unloaded either at its proper erection point or in a material yard near the central concreting plant.

The first operation was to erect a raised timber platform back of the Omaha abutment on which the traveler equipment could be assembled. Because the first span was on a curve and the second span crossed main-line railroad tracks, both of these spans were erected on falsework. For the remaining ten spans, however, the locomotive crane and the gin pole were brought into service. The steel towers and bents were assembled on the ground and raised into position on the concrete pedestals by the locomotive cranes. Then the trusses, which came from the shop in three sections, were assembled and lifted into position, using the 85-ft. gin pole at the outer end and the traveler at the inner end. The falls on the gin pole were operated by lines running to the drums of the locomotive crane. The trusses weighed about 20 tons each. With the trusses in place in one span, the traveler filled in the floor steel and advanced

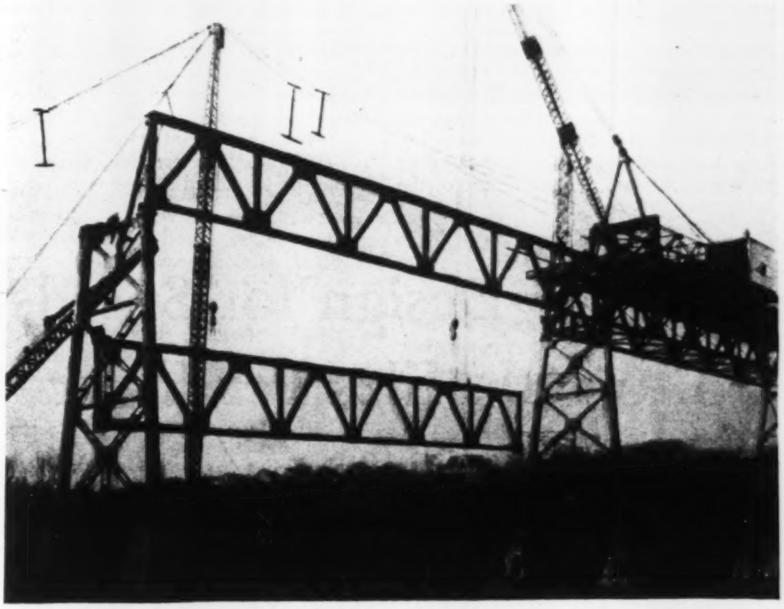


FIG. 8—A WEST APPROACH TRUSS being lifted into place by the traveler and a gin pole powered by the locomotive crane whose boom shows at the left.

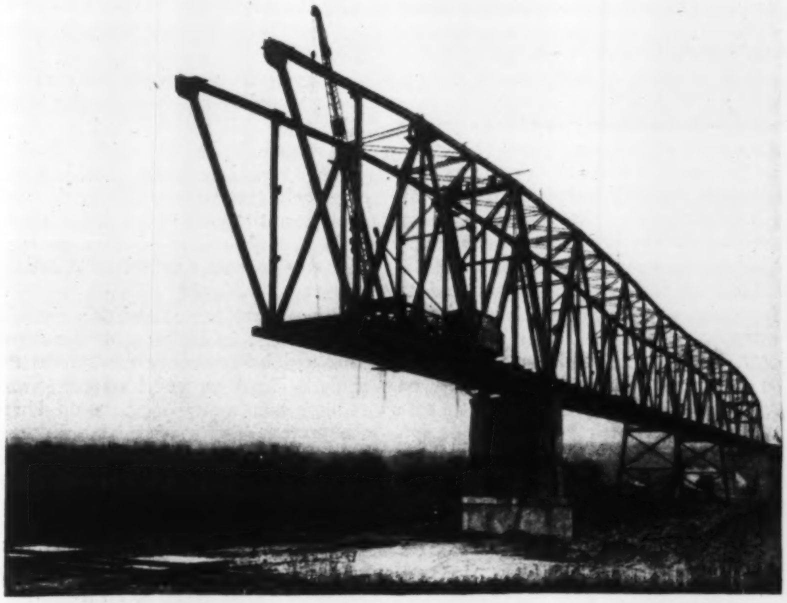


FIG. 9—CANTILEVER ERECTION of the second span of the 1,050-ft. continuous truss. In the left foreground are the timber pads on piles for supporting the erection bent at panel point L10'.

to erect the trusses of the next span.

For the structure east of pier 1 steel was brought up to the erection traveler along the deck of the completed structure. To accomplish this, a stiff-leg derrick was set up over the tower at pier B12 near the steel storage yard. The locomotive crane moved the steel within reach of this derrick, which raised it to deck level and loaded it on cars that were run forward to the traveler by dinkey locomotives. The traveler was so designed that the locomotive and the flat cars could run beneath it and out in front.

The two 525-ft. spans of the continuous truss were erected by the cantilever method. As shown in Fig. 8, the first four panels were erected on falsework, to provide a base from which cantilever erection could begin. Steel erection bents set on pile clusters were then installed, being placed at panel points L6, L10, L10' and L4'. (A fifth landing point was at pier 2 at the center of the 1,050-ft. span, panel point L16 of the truss.)

The only members that required additional material for erection were the floor beams over the erection bents and over the center pier, which had to take jacking ractions of 100 to 150 tons at the bents and of about 225 tons at the pier. Stiffeners and additional end connections were added to the four floor beams over the bents, while the extra large bending moment at the pier required a change of flange angles from 6x4x11 1/6-in. carbon steel to 6x6x$\frac{7}{8}$-in. silicon steel, also stiffeners.

Field riveting of the tension members was completed before any dead load was permitted to come into the member. Joints at which compression members abutted were not riveted until sufficient dead load had come upon the joint to bring the members into full bearing. Thus the lower-chord splices were completely riveted as erection progressed, while the top-chord splices were riveted only as the falsework behind them was released after the truss had been jacked to its true cambered position.

For each of the three 160-ft. deck-truss spans in the east approach, two falsework towers will be utilized. These towers, of timber, will occupy the third panel from each end of the truss, permitting the bottom chord to be erected in three pieces. Five erection pieces will comprise the top chord.

The South Omaha Bridge was designed and its construction is being supervised by Ash, Howard, Needles & Tammen, consulting engineers, Kansas City, Mo., for whom R. N. Bergendoff is resident engineer. Contract for both substructure and superstructure is held by the Kansas City Bridge Co., for whom C. W. Cubbage is general superintendent, assisted by C. S. Tupper, superintendent for steel erection. E. A. Wickham & Co., Council Bluffs, Iowa, was the contractor for the 5½-mile highway approach.

Highway Design for Speeds Up to 100 Miles Per Hour

Curvature, sight distance and superelevation are analyzed in their relation to maximum safe speeds on Oregon's highways, where main traffic arteries are designed for exceptionally high speeds

By **R. H. Baldock**

State Highway Engineer, Salem, Ore.

IMPROVEMENT in the design of automobiles has a tendency to make highways obsolete. This has been particularly true in recent years, and it applies most frequently to highways built a decade ago through rough country. Such routes are very likely to have standards of design, particularly in vertical and horizontal curvature, inadequate for safe travel at present-day vehicular speeds. Relocation of many such roads has become necessary, and for each case construction costs and the volume of traffic must be balanced to determine economical standards of construction. There are other cases, however, in which the engineer must decide on standards of construction largely on the basis of what speeds it will be safe and altogether desirable to provide for. In the hope of contributing something on such problems, the author has set down the following notes on observations and standards worked out for state highways in Oregon where the practice now is to design all trunk highways, except through mountains, for vehicular speeds of 75 to 100 m.p.h.

There is satisfaction in the fact that changes in the design standards of Oregon state highways, which began to be made several years ago as higher speeds were taken into consideration, quite definitely have been justified by the steadily increasing average speeds at which traffic is moving over major highways. For example, cars of this year's model, even those in the lowest price range, all are designed for speeds of at least 80 to 90 m.p.h. Braking power, too, has been considered so effectively that present-day stock model cars place the problem of designing for higher speeds squarely before the highway engineer who would build modern highways.

Oregon has recognized this situation by a change in highway design standards and by repealing the statute that made it illegal to travel state highways at more than a stipulated speed. Under the present law, in case of accident, a driver who was traveling at high speed may have to undertake the burden of proof in a controversy over cause; but high speed in itself is not illegal in Oregon, and automobiles travel daily over Oregon's highways legally and safely at speeds of 60 to 70 m.p.h.

In establishing standards of design for highway construction the ideal basis, of course, would be to make the design for a stipulated maximum speed. On a public highway, however, various types of traffic must be accommodated, and many factors enter into the conditions and limitations that must be met. In the following are outlined some of the major factors affecting the design of highways with relation to speed of vehicular traffic as these factors were studied and applied on the Oregon state highway system. Primarily, these are the selection of standards for width, grade, superelevation, horizontal and vertical curvature and easement on curves of both types.

Early in the consideration of vehicular speeds it was determined that three speeds would have to be considered, as follows:

1. Critical speed, the maximum that can be attained with the standards used and beyond which only the most skillful racing drivers can operate without extreme hazards.

2. Designed speed, 80 per cent of the critical speed and a speed that is safe for skillful drivers.

3. Recommended safe speed, which takes into account normal traffic conditions and the limitations of the ordinary driver. Hence, a speed somewhat less than the designed speed.

Regardless of the possibilities of further improvements of automobile design and the straightening of highways to eliminate hazards affected by the type of construction, there is a human element to be considered that places a definite limit on high speeds. This limit, however, even for the average driver, suggests safe speeds exceeding 50 to 70 m.p.h., depending upon the highway standards used. Therefore, to allow leeway for future improvement in the ability of the average driver and to provide a margin of safety for present

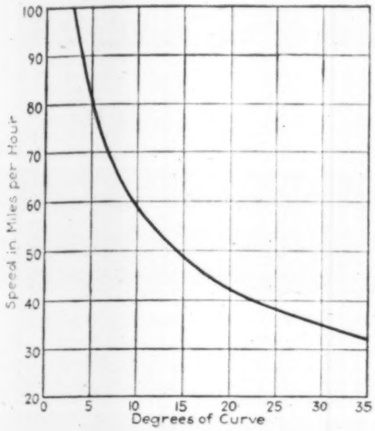


FIG. 1—LIMITATIONS on critical speeds imposed by curves up to 35 deg.

driving speeds, highway standards that permit speeds of 60 m.p.h. in the mountains and 100 m.p.h. in the valleys seem to the writer to be fully justified.

The critical speed of an automobile on a highway is controlled by the following factors: (1) the ability of the operator to function properly—the human equation, particularly in emergencies; (2) the ability of the mechanism of the vehicle to operate at high speeds without undue hazard; (3) the stopping distance or the distance traveled during the reaction time of the operator plus the braking distance; (4) the curvature, with which is involved easement, superelevation and tractive resistance—all of which are brought into the equation for centrifugal force; (5) the horizontal sight distance on curves, which, of course, varies with the curvature, the position of the vehicle on the road and the distance from the line of travel to the sight obstruction; (6) the sight distance over vertical curves; (7) the sight distance required in passing vehicles at varying speeds; and (8) the gradient used in the mountain sections.

The critical speed permitted by one factor for a certain standard will not necesarily govern for another factor, so, in order to correlate the standards, it is necessary to consider the effect on speeds of all the factors with relation to the standards used for both horizontal and vertical curvatures.

Curve design important

In studying horizontal curvature it is well to start with the basic formula for centrifugal force:

$$F = \frac{Wv^2}{gR}$$

In other words, centrifugal force varies directly with the weight of the vehicle, with the square of the speed and inversely with the radius of curvature. It is to be remembered that centrifugal force can be partly counteracted (1) by frictional resistance between tires and road surface, and (2) by superelevation. Moreover, the sudden development of centrifugal force at the point of curve may be lessened by the use of transition curves that cause the centrifugal effect to develop more gradually and thus to reduce centrifugal acceleration.

Research work done at Iowa State College (Iowa State College Bulletin, No. 120) shows that at high speed the frictional resistance between the tire and the pavement is of more practical importance in resisting centrifugal force than the superelevation of the roadbed. Superelevation is of necessity limited in amount, to avoid the possibility of slow-speed traffic slipping on ice-coated pavement. This consideration reduces the maximum superelevation to 0.13 ft. per foot of pavement width.

Oregon has adopted as the average superelevation for flat curves 0.075 ft. per foot and a frictional coefficient of 0.3. Using these values, the effect of frictional resistance is equal to twice that of superelevation. However, the importance of superelevation should not be minimized, since proper superelevation will overcome the lurch or side sway of both vehicle and passengers and at moderate speeds will balance the centrifugal force acting on them.

Contrary to common practice among highway engineers of regarding transition curves as a useless refinement, Oregon has used transition curves for about three years with very satisfactory results; the transitions are put on all curves. As far as centrifugal acceleration is concerned, there is no necessity for transitions on curves whose radii exceed 3,820 ft. 1°30′). For greater radii the maximum shift from circular curve to spiral curve is less than 0.83 ft. at speeds of 100 m.p.h. According to the formula, shift in feet equals

$$\frac{.0461\ V^6}{R^3}$$

where V is speed in miles per hour and R is radius of curve in feet.

For a 1-deg. curve the shift equals only 0.25 ft., which can be made easily in the traffic lane. Notwithstanding these conditions, the desirable features of transition curves in providing a suitable runoff for superelevation in eliminating the "rule-of-thumb" and "eye methods" used by engineers in effecting such runoffs partly on tangents and partly on curves, still make the use of transition curves a worthwhile practice. Furthermore, it materially improves the esthetic features of highway construction, especially when bridges are located on curves.

Some difference of opinion obtains as to the maximum rate of change in acceleration that will pass unnoted, *i.e.*, that will not cause a decided lurch of the car. Oregon experience indicates that for critical speeds an acceleration rate of 3 ft. per second per second can be used without discomfort, and this value has been adopted in laying out transition curves. As a result, the length of the transition curve is determined by the formula:

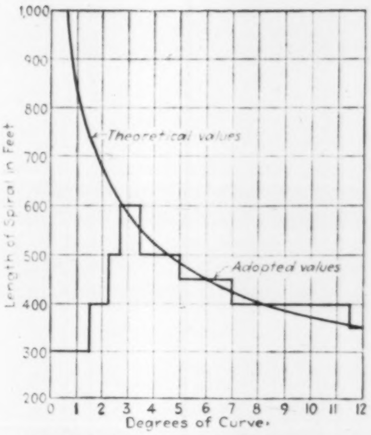


FIG. 2—LENGTHS OF SPIRALS required for critical speeds on curves up to 12 deg.

$$L = \frac{1.0517\ V^3}{R}$$

where V is in miles per hour, R is radius of the circular curve in feet, and L is the length of the transition curve in feet. This is the same as the formula which takes into consideration only superelevation required to meet a centrifugal ratio of 0.125 ft. per foot (which would apply in railway practice) plus the skid factor or tractive coefficient of friction that it is proper to include for highways and that changes the centrifugal ratio to 0.375 per foot.

The maximum rate of change in acceleration for critical speeds, noted in the foregoing, is reduced to 2 ft. per second per second for designed speeds and for recommended speed to between 1 and 2 ft. per second per second. The rather high rate of change of acceleration assumed for critical speeds does away with extremely long transition curves and results in a design that gives a change of acceleration not noticeable at the lower speeds and yet one that will allow a practical degree of protection for critical speeds.

Using the **3** ft. per second per second value, the permissible rate of change of acceleration for critical speeds on transitions is determined in Oregon practice by the formula:

$$V = \sqrt[3]{\frac{LR}{1.0517}}$$

where V is speed in m.p.h., L is length of the transition curve in feet, and R is radius of the circular curve in feet.

In Fig. 1 is shown the curve for critical speeds on curves of varying radii based on the superelevation standards used in Oregon, using a coefficient of friction of 0.3. The curve shown in Fig. 2 illustrates the length-of-transition curves required to meet the same critical velocity as given in Fig. 1.

RELATION OF CRITICAL AND DESIGNED SPEEDS TO SOME LIMITING FACTORS

Oregon Standards for Heavy-Traffic Roads

Type of Country	Deg. of Curve	Vert. Curve Sight Distance, Ft.	Critical Speed, M.P.H. Wet Pave.	Critical Speed, M.P.H. Dry Pave.	Designed Speed, M.P.H. Wet Pave.	Designed Speed, M.P.H. Dry Pave.	Stopping Dist. in Feet (Reaction Time Plus Braking Dist.) Critical Speed, Wet Pave.	Stopping Dist. Critical Speed, Dry Pave.	Stopping Dist. Designed Speed, Wet Pave.	Stopping Dist. Designed Speed, Dry Pave.	Sight Dist. on Curves, Ft.	Sight Dist. for Passing Cars, Ft.	Dist. to Horizontal Obstruction,* Ft.	Dist. to Vert- l Obstruction, Ft.
Valley Sections....	2	1,500	100	100	80	80	740	490	490	330	960	860	40	750
	3	1,500	100	100	80	80	740	490	490	330	785	860	40	750
	4	1,500	90	90	75	75	600	400	430	290	675	860	40	750
Rolling Hills......	6	1,000	75	75	60	60	430	290	290	190	450	750	27	500
Mountains........	10	1,000	58	58	45	45	275	180	175	120	355	730	27	500
Tangents..............		1,500	100	125	80	80	740	740	490	330	...	860	..	750

*In other words, distance from center line of highway to building or other obstruction to view ahead.

In Figs. 1 and *2* the formulas used are:

$$\text{Fig. 1, } V = \sqrt{\frac{R(s+f)}{0.067}}$$

$$\text{and Fig. 2, } L = 60.6R^{1/2}(s+f)^{3/2}$$

The symbols are defined as follows: V = critical speed in m.p.h.; R = radius of curve in feet; s = superelevation of main curve in feet per foot of roadway width; f = skid factor (assumed as 0.3); and L = length of spiral in feet.

In Fig. *2* no curves sharper than 12 deg. are shown because highways of major importance are here being considered. For roads of lesser importance, designed speeds are lower and shorter spirals are used. The broken line in Fig. *2* indicates the spiral lengths actually used. Spiral lengths increase (moving from right to left on the diagram) as permissive critical speeds increase with flatter curves up to about 3 deg. Here a permissive critical speed of 100 m.p.h. is reached, and beyond this point—*i.e.*, for all flatter curves—lengths of spirals are governed by radius of curvature alone and are determined by the formula:

$$L = \frac{1.0517\,(100)^3}{R}$$

For calculations involving stopping distance when using braking power, Oregon standards assume that the coefficient of friction available for stopping is 0.5. This is the rate for wet concrete pavement, which gives a stopping rate of 16.1 ft. per second per second. This rate is considered the maximum rate of deceleration reasonably safe and comfortable. Some allowance for reaction time of the driver must be included, and this is taken as one-half second. The formula then becomes

$$S = \frac{V^2}{30f} + 0.73\,V$$

where S equals stopping distance in feet, V equals velocity in miles per hour, and f equals coefficient of friction available when stopping.

The accompanying table divides main-traffic highways of the state into three groups: valleys, rolling hills and mountains. For these three classifications the table gives relationships between the various values that should be considered in determining critical and design speeds. For this table the minimum critical speed was computed with relation to the varying factors based upon the velocity formula on curves and the stopping distance in relation to the horizontal and vertical sight distance. On valley sections the ruling curves are 2 deg., which permits safe passing. Curves as sharp as 4 deg. are used only when cost is excessive. Maximum grades are 5 per cent, except that 6 per cent may be used for lengths not exceeding 3,000 ft.

As a convenience in planning and laying out Oregon highways, somewhat similar tables have been constructed (not reproduced herewith) in which are tabulated limiting speeds under various conditions of the limiting elements. In other words, tables which show at a glance that in valley sections with flat curves the limitation on critical speed would be that imposed by the safe stopping distance and not that imposed by reason of sight distance, spiral length, superelevation or tire friction. The critical speed for sight distance in a typical case is 103 m.p.h., and for the several other factors 125 m.p.h.; hence in this instance sight distance governs. Where the critical speed works out as 103 m.p.h., as a safety factor the designed speed is reduced *20* per cent, or to 80 m.p.h., and the recommended safe speed under normal traffic conditions becomes 60 to 70 m.p.h. This last-named range also appears to be the best speed for passing vehicles. It is to be noted that within certain limits increases in speed decrease hazards of passing vehicles traveling in the same direction.

The safe distance required for passing a car moving in the same direction at a time when a car is approaching from the opposite direction involves three factors—namely, the speeds of the three vehicles concerned. A different set of curves must be used for each speed assumed for the vehicle approaching from the opposite direction. In Fig. 3 the curves are drawn for an 80-m.p.h. speed of the approaching car. The formula from which these curves were drawn is

$$X = (S + S'')\left(\frac{A}{S - S'} + t\right)$$

where X = sight distance in feet required for safe passing; S = speed of passing vehicle; S' = speed of vehicle being passed; S'' = speed of vehicle approaching from opposite direction—assumed in this instance as 80 m.p.h.; A = distance in feet from back of car passing to front of car being passed, at time decision as to passing is made, assumed in this instance to be 50 ft. (This is safe, ordinarily—more is needed if long truck bodies are involved.) t = time, in seconds, required for passing vehicle to return to right-hand lane after clearing vehicle being passed. Assumed in this instance to be 3.0 sec., which is *220* ft. at 50 m.p.h. (All speed values in ft. per sec.)

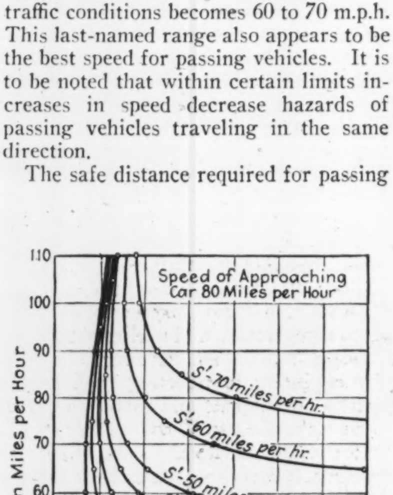


FIG. 3—SIGHT DISTANCES required for safe passing at speeds of 30 to 110 miles per hour.

In valley sections, on curves as well as tangents, the various features of design permit critical speeds of 90 to 103 m.p.h., designed speeds of 75 to 80 m.p.h., and recommended speed under normal traffic conditions of 60 to 70 m.p.h. On two-lane highways passing on curves is not safe where curvature exceeds *2* deg. Through rolling hills normal traffic permits speeds of 50 to 60 m.p.h. with safety, and through mountainous country 40 to 45 m.p.h. When the traffic reaches the point of congestion that makes it impossible to attain a ruling speed equal to the recommended safe speed, consideration should be given to widening two-lane pavements to four-lane widths as the latter permit increased speeds with the same safety factors.

Oregon practice makes two-lane highways on a 40-ft. roadway, of which 20 ft. is paved and flanked by 6-ft. gravel shoulders; four-lane highways are built on 66-ft. roadway widths having pavement 44 ft. wide. The pavement is marked into 10-ft. lanes with a 4-ft. neutral zone at the center to separate traffic moving in opposite directions. The present tendency toward high speeds indicates the desirability of increasing the width of lanes to 11 or 12 ft. each.

Diversion-Tunnel Driving at Fort Peck Dam

Four parallel tunnels having a lined diameter of 26 ft. and averaging 6,317 ft. in length have made a good start during the late fall and winter

TUNNELING had just been well begun in November, 1934, a year after the first construction laborers were put to work at Fort Peck. Besides recording the progress then made, which included getting the tunneling plant onto the ground, opening up the approach cuts and getting the concrete plant ready, the present article has included some of the winter operations as they have been reported since last fall. While this is not adhering strictly to the title that was set for the Fort Peck construction review articles, of which this is the second, it does wind up loose ends and permits a more rounded account than would have been possible if the writing had held precisely to nominal dates. The final story of this tunneling operation will come as the work advances well into enlargement of section but meanwhile the present account of rock conditions and pilot tunneling methods is a record of interest. —EDITOR.

TUNNEL CONSTRUCTION at Fort Peck, as stated in the general review of construction operations (*ENR*, May 9, 1935, p. 659) ranks as a major unit of work because of the large excavated bore of 32 ft. and the aggregate length of over 4.8 miles. Considered as a tunneling task alone, the controlling construction condition was the difficult material, a dense marine shale that disintegrates rapidly on exposure to the air. There was a construction advantage because the four mile-long tunnels were parallel and close together and attack could be concentrated at the portals.

Summarized, the outstanding construction features to be noticed are: (1) asphalt coating of tunnel rock to delay disintegration; (2) rock saws operating on the principle of a coal cutter, capable of making a circular cut 32 ft. in diameter; (3) muck handling and spoil disposal by belt conveyors.

Tunnel structure

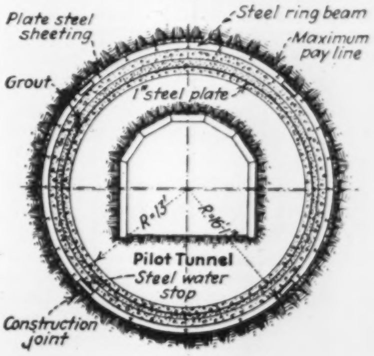
The essential elements of tunnel structure are given by the cross-section, Fig. 2. As shown, the finished diameter of the four tunnels having an average length of 6,317 ft. will be 26 ft. The respective lengths are 5,379, 6,006, 6,629 and 7,254 ft. on a total of 25,268 ft., or a little over 4.79 miles.

The capacity of the four tunnels under full-reservoir conditions at El. 2,250 ft. is 84,000 sec.-ft. The average river discharge is 8,000 sec.-ft., and that of the maximum flood of record (1908) was 154,000 sec.-ft. Thus the 254,000-sec.-ft. spillway (*ENR*, May 9, 1935) will have to go into action when about half the record flood has been reached, but combined the tunnels and spillway will carry over twice the record flood. An additional safety factor is that flood peaks will be flattened by the use of the top 8 ft. of the reservoir for flood-control storage.

As noted, the rock bore for the tunnel will be 32 ft. in diameter. This will be supported with steel ring-beams, light I-beam purlins and steel-plate sheeting. The tunnel will be lined first with 21 in. of concrete, inside of which will be a 1-in. steel lining or pipe with a 9-in. reinforced concrete interlining. Because of the rapid disintegration of the shale the concrete is required to be placed against the rock within twenty days after excavation is completed. After the steel shell is in place, grout will be forced back of it, to insure complete filling of the space between the steel and the concrete.

From the top of the bluff, about 2,350 ft. from the inlets, a main control shaft 60 ft. in diameter and an emergency gate shaft 16x34 ft. in plan are being sunk about 250 ft., to intercept each tunnel near the axis of the dam. Each control shaft will be braced with steel and then lined with 5 ft. of heavily reinforced concrete containing a steel shell varying from ⅝ to 1⅛ in. thick. A concrete chimney, with ring gates, inside the main control shaft will control the flow of water after the dam has been completed. Gates probably of the stoney type will be installed in the emergency shaft chamber, which is located 72 ft. upstream from the control chamber. The walls of both shafts are heavily reinforced, to take the full load

FIG. 2 — TUNNEL CROSS-SECTION where steel supports are required. Lining strength and impermeability of unusual character were sought in designing the four tunnels.



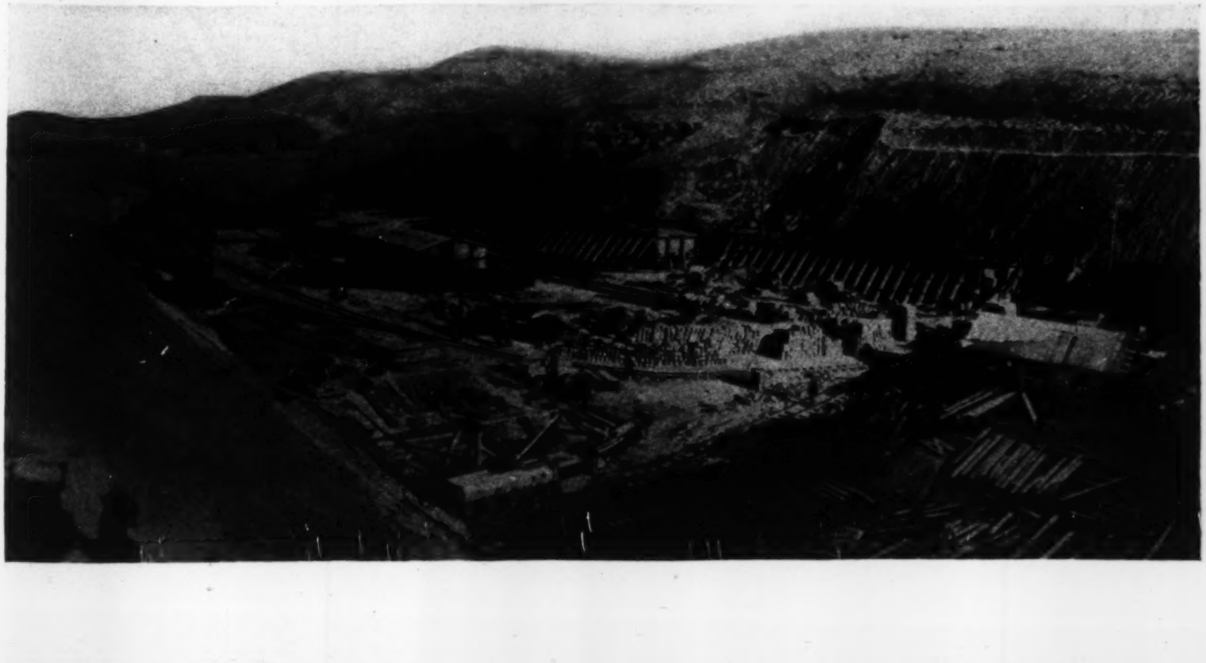


FIG. 1—OPERATIONS at one of the approach cuts to the four portals of the tunnels for river diversion and reservoir control at Fort Peck Dam.

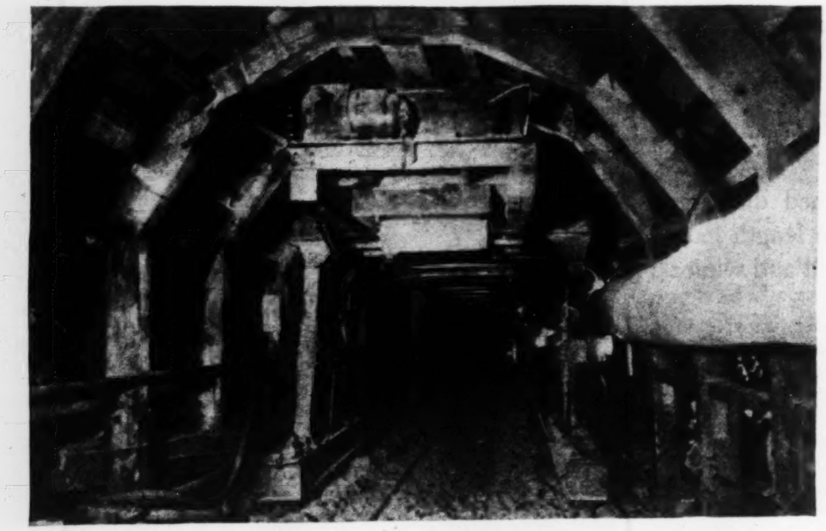


FIG. 3—INSIDE A PILOT TUNNEL where the belt conveyor on gantry carriages straddles the muck-car track and loads the cars directly from the machine mucker at the heading.

of the surrounding shale. Intake and outlet structures are heavy concrete; together they require about 197,000 cu. yd., mostly in heavy headwalls to hold the cut face of the bluff.

Tunnel excavation

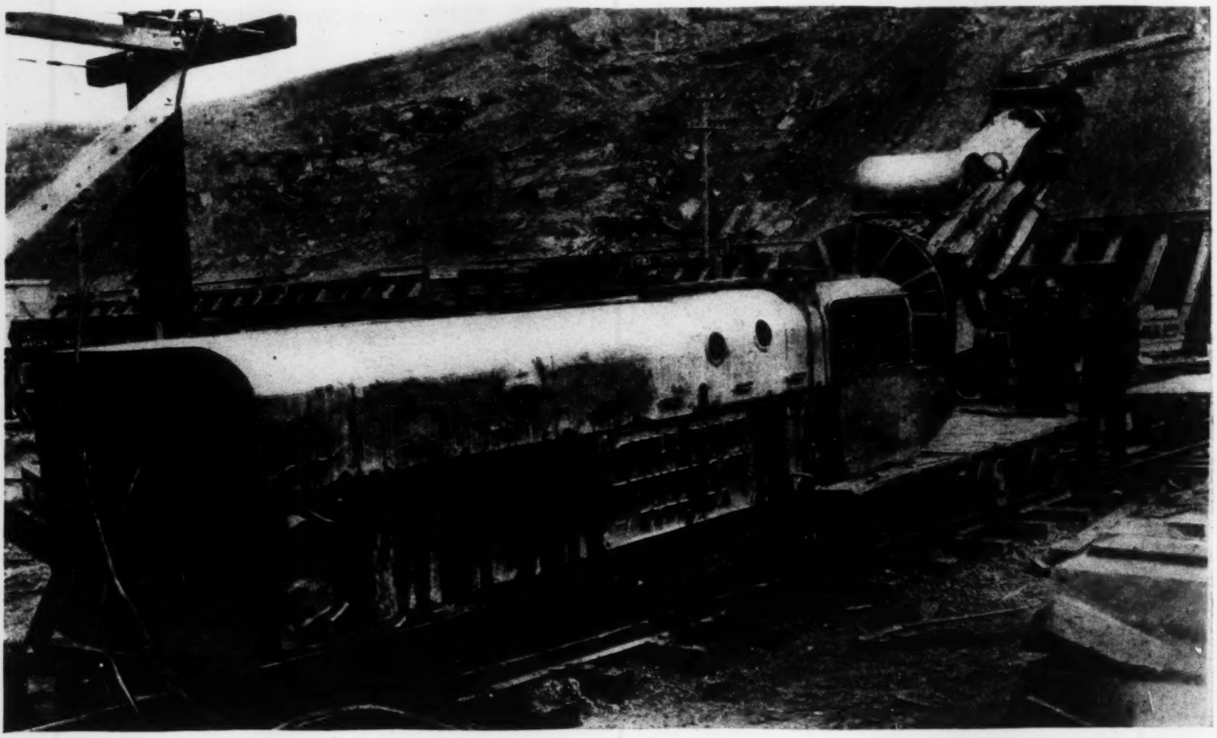
The controlling conditions of tunnel excavation were the large diameter of bore and the rock, which disintegrated rapidly. At the time the work was observed the methods developed to meet these conditions were in full operation except the unique equipment of rock saws, which is dscribed later. Conclusions at this time concerning methods and equipment are limited, therefore, to the experience had very early in the tunneling operations.

Considering first the rock conditions: The formation is shale, seamed with bentonite and known geologically as bear-paw shale. The bentonite is a compacted volcanic ash, which, when wet, is as slippery as soap. Both bentonite and shale are very impervious, but the formation is much faulted; some of the faults show a displacement of 68 ft. These faults have been carefully located and when encountered in the tunnels extra bracing and sometimes grouting are required. The shale has a high moisture content, and it is to keep this from escaping, and thereby breaking up the rock, that the face immediately after shooting is covered with at least three spray coats of cutback bituminous material. As an additional precaution the air in the tunnels is kept to at least 90 per cent relative humidity by means of combined air and water jets spaced 200 ft. on centers. Water is delivered under 125-lb. pressure, and the air line is tapped off the supply line for the air drills, in which pressures range from 90 to 100 lb. per sq.in.

Tunneling was begun by driving in each bore a pilot tunnel 16⅓ ft. wide and 14 ft. high. Headings for the pilot tunnels are driven in the ordinary way, using 1⅞-in. air-driven auger drills in 42 holes, 9 ft. deep. The holes are loaded with 30 per cent powder, and the advance is 8 ft. per round, accomplished on the average in six hours. Powder consumption is 0.95 lb. per cu.yd. of muck. After the shooting, all loose material is barred down and careful watch is kept of any subsequent loosening due to disintegration. Spraying with the bitumen is started immediately, and timbering is brought up to the face. All tunnel workers use hard helmets.

Fresh air, brought in through a 22-in. canvas pipe, is delivered continuously

FIG. 4—A ROCK-SAW operating after the fashion of a coal cutter and designed for section enlargement of Fort Peck tunnels by making a circular cut around the perimeter of the bore.



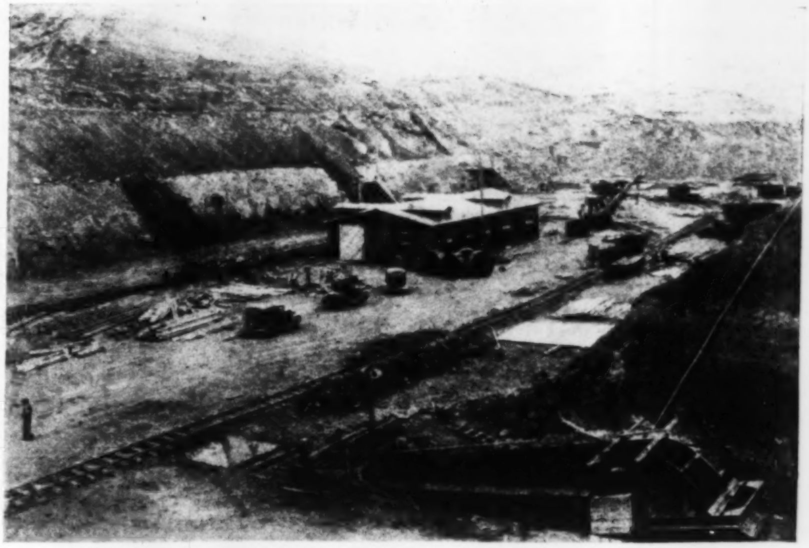


FIG. 5—UPSTREAM PORTALS; tracks from tunnels lead to hopper (in foreground) from which materials are carried by apron conveyor to a crusher, thence on belt conveyor to muck piles.

at the heading by fans at the portal at the rate of 4,500 cu.ft. per minute. Air for drills is compressed at a station near the lower portal by two compressors driven by 450-hp. and 225-hp. motors. It is piped under 100-lb. pressure to all tunnel headings and to the inlet, outlet and shaft operations.

Close to the inlet portals square sets with kneebraces of 10x10-in. timbers were used close together. Farther in, the pilot tunnels (five-segment arches 3 to 8 ft. on centers) are used with 2x12-in. open lagging. Posts sunk into the floor a few inches are set straight. Each collar contains 292 ft.b.m. of lumber, collar bracing amounts to 11 ft.b.m., and lagging to 30 ft.b.m. per foot of tunnel.

Mucking is done with 75-hp., ½-yd. overthrow machines discharging onto belt-conveyor loaders 60 ft. long, which accommodate five 6-yd. dump cars under the 30-in. elevated belt. The two gantries supporting the belt (Fig. 3) are 40 ft. apart and move on a track of 8-ft. 2-in. gage, straddling the muck-train track of 3-ft. gage. The muck-train track leads out of the tunnel direct to the crusher hopper at the loading end of the spoil-conveyor system. Trains of five cars each are hauled by electric locomotives, utilizing a combination of trolley and battery.

At the time of inspection, Nov. 3, 1934, the pilot tunnels had advanced 340, 182, 278 and 1,042 ft. A report from the work as of March 1, 1935, showed advances of 4,169, 4,281, 4,766 and 5,222 ft. From Jan. 24 to Feb. 23 a claimed world's record for speed was set by pilot No. 3, when the heading was advanced 1,508 ft. in 89 eight-hour shifts, or an average for the month of 16.9 ft. per shift. On Feb. 23 the 24-hour advance was 65 ft.

In closing comment on the pilot-tunnel excavation, it should be noted that rock saws (Fig. 4), adapted from coal saws, have been brought to the job by the contractor for use on the enlargement, but due to the small section of the pilot tunnel and to the lack of sufficiently trained operators the saws have not been used except in an experimental way. It is hoped eventually to obtain more speed by using them, to save on powder and drilling and to avoid overbreak in the enlargement. The saw was used on tunnel 4 on the pilot section. It cuts a slot about 9 ft. deep and 4 in. wide, following an arc around the circumference of the section. It operates over a track and is powered by a 135-hp. motor.

Conveyor spoil disposal

At both inlet and outlet the excavation of overburden and shale rock from 25 to 65 ft. deep is by diesel shovels and draglines loading trucks (some of which are of 16-cu.yd. capacity), which discharge into a train of belt conveyors that carry the material to spoil banks some 3,000 ft. away. To break up the larger rock, the material passes through crushers to reduce it to conveyor-belt size. At the inlet the 30-in. conveyor system is made up of 500-ft. sections, a spreader with a 60-ft. swinging boom at the end and a 70-ft. tail piece. The spreader and tail piece are mounted on rails with a 16-ft. gage. This system raises the material from about El. 2030 at the inlet forebay to about El. 2160 at the present spoil-bank level. The dumped spoil is producing a 60- to 70-ft. fill with a face slope of 1 on 1½. All muck from the pilot tunnels is also disposed of over this conveyor system. Up to March 1, 100,000 cu.yd. of outside excavation and 160,000 cu.yd. of tunnel muck had passed over the belts.

At the outlet portal a similar installation of 36-in. conveyors 2,950 ft. long had handled 300,000 yd. of wall and raceway excavation up to March 1. This lower train of conveyors has handled a maximum of 10,000 yd. per day. The belt speed is 425 ft. per minute. A 4½-ft. sheep-nose toothed crusher operated by a 150-hp. motor reduces the material to belt size. The material is fed to the crusher by a steel apron conveyor 6 ft. wide. The conveyor system is made up of sections varying from 200 to 900 ft. long. A stacker at the end is crawler-mounted and consists of a 100-ft. tail piece, which can telescope 50 ft., and a swinging 70-ft. boom.

Concreting plant and methods

At the time of inspection, tunnel concreting had been confined to the intake and outlet structures. It is found impracticable to maintain artificially a high humidity near the shale in open-cut work, but sealing solution is applied within three hours of exposure, and concrete is poured as soon after that as conditions will permit.

As soon as the key excavation is completed, steel and forms are placed and a pour is made up to the top of the slab in the raceway. A depressed key with dowels is placed in the top of this pour. During the winter, it is reported, it was necessary to protect the surface of this concrete with tarpaulins and to use steam heat for periods ranging from five to ten days, depending upon the severity of the outside temperature. At the end of this period the forms for the second lift, extending up to within 6 ft. of the top of the wall were placed, and the concrete was poured and given immediate protection against cold weather, a similar key with dowels being inserted at the top of this pour. The top 6 ft., which contains a channel for a conduit, is poured last. The forms are of wood, standardized in 10-ft. panels wherever practicable. The concrete is delivered to hoppers in bottom-dump buckets at the rate of about 40 cu.yd. per hour, from which it is handled by chute and elephant trunk to its place in the form. A pumpcrete machine is also in use at the lower portal. The wall is constructed in alternate 20-ft. and 40-ft. blocks, which are keyed and doweled together. In addition, an 8-in. by ¼-in. steel plate is placed in the construction joint as a waterstop, and provision is made for calking all joints on the raceway side.

Since there is a total of 225,000 cu.yd. of concrete in the outside structures at inlet and outlet, plus 375,000 cu.yd. in the tunnel and shaft lining, a large concrete mixer plant has been installed in the outlet raceway equipped with three 2-yd. mixers, over which is located aggregate-weighing equipment. A smaller plant will later be erected at the inlet end.

An aggregate storage of 130,000 cu. yd. was built up prior to the winter season, as it must be hauled in from a

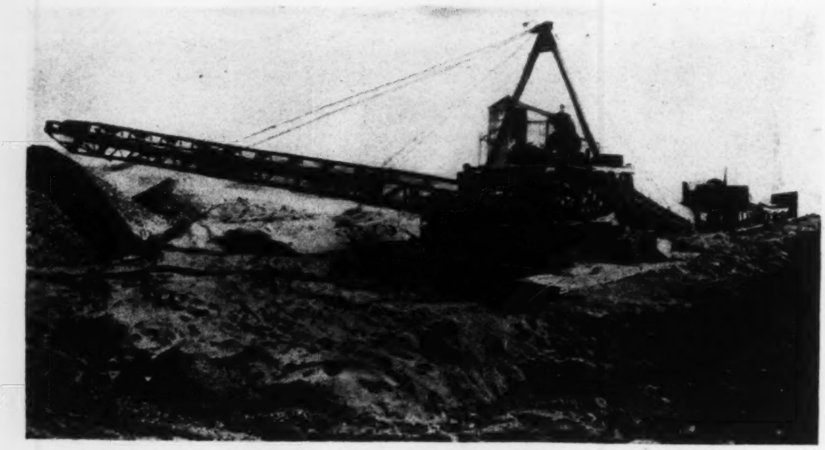


FIG. 6—THE STACKER with 50-ft. boom followed by a 100-ft. tailpiece telescoping 50 ft. at end of conveyor system at downstream portals is tractor-mounted.

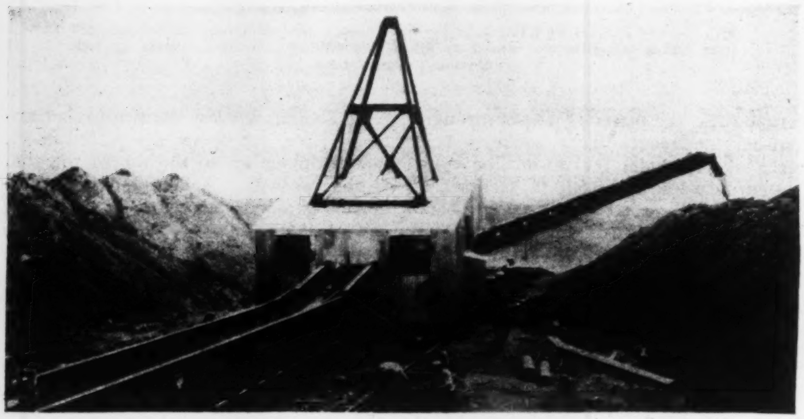


FIG. 7—THE SPREADER with 70-ft. boom at end of 3,000 ft. of conveyor system for upstream portals, together with its telescoping tailpiece, is mounted on rails. Note housing to permit winter operation.

washing and screening plant located at Cole, 70 miles away. During the fall, aggregate was delivered at the rate of 100 cars per day. The gravel comes in two sizes, ¾ to 1½ in. and ¼ to ¾ in. The oversize is utilized for gravel filling for the toes, on which 750,000 cu.yd. were dumped into place last fall. Aggregate at the storage pile is unloaded from bottom-dump cars at the foot of a belt-conveyor system onto a flat feeder chute vibrated with a solenoid to feed evenly onto the belt, which is 36 in. wide and carries the aggregate to another belt 604 ft. long, running over the top of the stockpile. The elevating conveyor rises at an angle of 16 deg. to a height of 65 ft. and has a capacity of 300 tons per hour. It runs at 350 ft. per minute. A tripper distributes the aggregate into one of three bins formed by transverse timber bulkheads. The bulkheads extend up 25 ft. only and are made of 2x12-in. planks double-posted with 6x10-in. timbers spaced 6 ft. on centers. The total height of the stockpile trestle to the base of rail of the tripper is 65½ ft.

Material is retrieved from the stockpiles by belt conveyors operating in two 9x9-ft. tunnels, into which gate-controlled hoppers discharge at 28-ft. intervals, so that the stockpile may be kept at even heights on the two sides of the bulkheads. To prevent the material from arching over the discharge hoppers, a perforated pipe A-frame is supplied with compressed air under 100-lb. pressure. Air under pressure is also used in a pipe nozzle, to prevent arching in the bottom-dump cars as they are unloaded.

At the end of the tunnels are two elevating conveyor belts leading to the aggregate hoppers over the mixers. The sand hoppr holds 150 cu.yd., and the gravel hoppers hold 150 cu.yd. each. By adjusting gates in the hoppers at the discharge end of the belts, aggregate can be delivered from either conveyor to the appropriate hopper.

Cement comes in bulk and is unloaded by a cement pump to any one of three steel tanks, which have a combined capacity of 25,000 bbl. The pump can also deliver direct to a 500-bbl. storage bin over the mixers. The cement is normally recovered by screw conveyor under the silos, feeding a bucket elevator to the elevated storage bin. The capacity of the bucket elevator is 250 bbl. per hour.

Water is supplied by a 6-in. cast-iron pipe laid by the government across the river and is connected to the filtered-water supply of the construction city on the west side of the river. A 150,000-gal. tank is located on the hillside for emergency use.

Progress and direction

Since May 6, 1934, more than 1,600,000 yd. of overburden and shale, to give access to the tunnels, portal structures and shafts, have been moved by the contractors, the Silas Mason Company, Ind., and the Walsh Construction Company, who have the diversion tunnel contract at a bid price of $7,300,000. Excavation has been practically completed at both portals, and construction of the concrete structures is well under way, including the open-cut section of the tunnels. Work on the wall structures at the tunnel outlets continued throughout the winter except for a few days in the most severe cold. Work has progressed all winter in the pilot tunnels. Pilot shafts 8x8 ft. have been sunk in all eight of the control and emergency shafts. The enlargement was started recently.

The Fort Peck project is being constructed under the supervision of Major T. B. Larkin, district engineer, U. S. Army, with Major Clark Kittrell as chief of the operations division, which has charge of all construction activities. Captain A. W. Pence is directly in charge of the tunnel work.

Plans for the project are all prepared in the Kansas City office of the U. S. Engineers, Missouri River Division, under the supervision of Col. R. C. Moore, division engineer.

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Paving Brick Standards Modified

Two varieties were eliminated, and two others were added to the six recognized varieties of paving brick at the annual meeting April 12, 1935, of the permanent committee on simplification of variety and standards for vitrified paving brick of the Department of Commerce. The added varieties were both vertical fibre lug brick, one 2½x4x8½ in. and the other 3½x4x8½ in.; they replaced the preceding 4x3½x8½ in. and 3x3½x8½ in. wire-cut lug (Dunn) brick varieties. The six recognized varieties are now as follows:

Variety	Size in inches	Percent of 1934 shipments
Vertical fibre lugless...	2½x4x8½	9.7
Vertical fibre lugless...	3x4x8½	5.9
Repress lug	4x3½x8½	9.6
Vertical fibre lug......	2½x4x8½	19.1
Vertical fibre lug......	3½x4x8½	6.6
Vertical fibre lug......	3x4x8½	37.5

These varieties amounted to 88.4 per cent of the total shipments in 1934.

Sewage-Treatment Processes Aided by Long Outfall

Agitation and aeration in 3½-mile outfall line to supplement purification provided by aeration and sedimentation in the plant at St. Albans, Vt.

By Henry W. Taylor
Consulting Engineer, New York, N. Y.

TO RELIEVE gross sewage pollution of a small stream flowing through a dairy farming country, the city of St. Albans, Vt., has undertaken the construction of a sewage-treatment plant and a long outfall sewer. The treatment plant will provide pre-aeration, sedimentation, post-aeration, and considerable additional aeration accompanied by thorough agitation will take place in the outfall sewer.

In the fall of 1933 the city was faced with threatened suits by farmers who were not permitted to deliver the milk from their farms to certain municipal markets, and it was decided to take advantage of PWA facilities to construct a sewage-works. The plant is now under construction, and will soon be put into operation. Present interest is confined to the basis of design for the development, which had to meet various local conditions that materially affected the type of project.

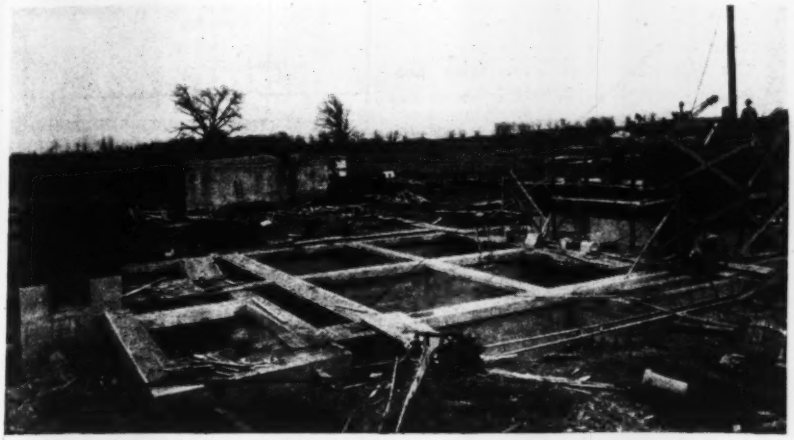


FIG. 1—SEWAGE-TREATMENT PLANT for St. Albans under construction. This view includes in the foreground the post-aeration and settling tanks and grit chamber. The pre-aeration unit is under the laboratory building being erected at the right. In the background is the sludge-digestion tank.

Existing conditions

The city of St. Albans is situated east of St. Albans Bay of Lake Champlain and has a population of about 9,000. A combined sewer system was installed about 40 years ago, and the trunk sewer discharges into a small ditch known as Gas Brook. This stream flows in a northerly direction, then loops around to the south and discharges into St. Albans Bay. From the outlet of the trunk sewer to the mouth of the stream the flow distance is about 6½ miles. The stream is practically dry in the summer time, and the sewage discharged into it from the city is undiluted during dry weather for the first mile and a half, and at no time is it diluted in any considerable ratio until it reaches the backwater from the bay, which extends about 1 mile upstream in dry weather and about 2 miles at high-water stages of the lake.

The purification of this practically undiluted sewage by nature has been remarked by all those who have inspected the stream and followed its course. These natural processes have been highly effective from the physical angle as far as the mouth of the stream is concerned. However, nature is no respecter of persons or personal property, and this natural disposal has taken place on farmlands traversed by the stream for 5½ miles. But the sanitary conditions along the course of this brook can easily be imagined and need no detailed description.

There has been recurring friction between the farmers and the city over this situation for a great many years. About fifteen years ago a protest of the farmers was allayed by the purchase of a site for a sewage-treatment plant, but the projected plant was not built. However, in 1933, milk inspectors took definite action with reference to certain farms through which this stream flows and temporarily refused to accept, for certain markets, the milk produced. This naturally incensed the owners of the farms, and suits against the city were threatened.

Due to the fact that few, if any, municipal sewage-treatment plants have been installed in this vicinity, there was even more than the usual aversion to spending money for such purposes, and consequently the taxpayer's eye was focused unrelentingly on items of expenditure for maintenance of a new public utility. This question of maintenance cost, unless properly met, would have made unlikely a successful vote on bonds for the construction of the project. The initial cost was also a matter of serious concern.

To discharge a treated sewage into this same stream, which would continue to flow through the same farm property, would, if trouble were to be avoided, require a plant of faultless design combined with faultless operation. Both of these requirements would have been difficult to meet, particularly in view of the unlikelihood of securing support for the construction of a plant with high maintenance cost and of obtaining thereafter yearly funds for the continuance of such maintenance.

Basis of design

Giving these considerations their due weight, there seemed only one conclusion to be drawn and one fundamental basis of design of the project: take the sewage out of the stream. This basis of design completely satisfies the legal requirements in the case. Furthermore, disposal in another body of water of larger proportions would reduce the degree of treatment required and meet the need for rigid economy in design. It also would avoid the danger of inadequate operation that would go with an elaborate and expensive plant.

Except for the assistance given by the local topography, ways and means for executing this basis of design might have proved somewhat difficult. However, it was found that there was a possibility of running the outfall sewer on a straight line for about 16,000 ft., and that it would then avoid deep cuts and at the same time provide a drop in elevation of 264 ft. to the level of the backwaters of Lake Champlain. This alignment entirely avoided the dairy farm country and formed what might be considered the south side of the stream loop. In other words, the sewage effluent will be detoured through the outfall sewer to a point beyond the range of dairy farming and to a swamp that is hardly accessible in the summertime but adjacent to backwater from the lake and always flooded in the spring to a depth of 4 or 5 ft.

In view of the circumstances, the de-

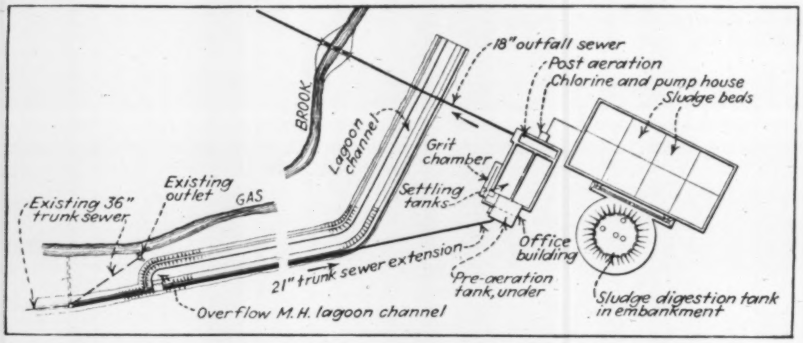


FIG. 2—PLANT PROCESSES include screening, grit removal, pre-aeration, sedimentation and post-aeration. Stormwater overflow is retained in lagoon until flow in sewer subsides and is then passed through the treatment plant.

sign adopted for treatment provides preliminary treatment and aeration at a new plant and additional aeration during the flow through the outfall sewer. The purpose is to produce an effluent that, when discharged into the stream again, will be in better condition than it would have been after a 6½-mile trip in the natural water course.

Some consideration was given to providing a new outlet into St. Albans Bay, but this idea was rejected, first, because of the additional cost that would be involved and, second, because it would introduce a new psychological element by producing a new point of sewage discharge. It was decided to discharge the sewage effluent into the same water course that had received the raw sewage for the last 40 years, but to discharge it at a point that elminated sewage flow through the dairy district that had been suffering damages from former conditions. This policy involved improvement of conditions but no new outfall conditions.

Sewage-treatment plant

The sewage-treatment plant and outfall sewer combined must necessarily secure two results. The treatment plant itself must primarily prevent the formation of sludge banks in the mile of stream between the point of sewage discharge and St. Albans Bay. The plant must also reduce the biochemical oxygen demand at least as much as nature had reduced it through natural processes. During certain times of the year it was also thought desirable that the sewage effluent should be disinfected for the protection of sporting activities on St. Albans Bay.

To accomplish these results, the treatment plant consists of the following elements: the sewage passes first through a coarse screen (with a screening shredder), which is followed by a pre-aeration tank designed to remove sulphide gases, satisfy the immediate oxygen demand and produce a certain amount of flocculation to aid subsequent sedimentation. The sewage passes from the pre-aeration tank to shallow flat-bottom settling tanks, each provided with longitudinal and transverse conveyors delivering sludge to a single sludge hopper in each tank unit. The settled effluent passes to a post-aeration tank in which the oxygen demand is still further reduced, and a supply of oxygen is introduced before the sewage starts its trip through the outfall sewer. The sludge is decomposed in an unheated sludge-digestion tank, with fixed cover.

The following data summarize the principal elements involved:

1. Coarse screenings
    - Shredder for screenings
2. Pre-aeration; 10 min.; 0.05 cu.ft. of air per gal. of sewage
    - Skimming chamber
    - Chimney for exhauster
3. Sedimentation; 1.5 hour
    - Longitudinal and transverse sludge conveyors
    - Effluent weir back from end of tank
4. Sludge digestion; unheated tank; 2 cu.ft. per capita
    - Sludge gas heater, etc.
5. Post-aeration; 20 min.; 0.1 cu.ft. of air per gal. of sewage
6. Outfall sewer
    - Mechanically ventilated for 6,000 ft.
    - Mushroom aerator and vent stack
    - Surface absorption; drop manholes; high velocities; ventilation
    - Length of travel, 16,000 ft.
7. Chlorination; summer, 12 p.p.m.

Total period of treatment, including tanks, aeration, and outfall sewer, about 4.5 hour.

Outfall sewer

The outfall sewer starts out as an 18-in. pipe laid on a 0.25 per cent grade. This first section is about 6,000 ft. long and traverses a flat plateau where it would be impossible to provide drop manholes or any other form of agitation without increasing the cost of the project considerably. It is ventilated by an exhaust blower, which changes the air every half hour. Surface absorption of oxygen will obtain in this section of the outfall sewer, and any gases will be quickly removed.

The second section consists of an inverted siphon, at the lower end of which is a mushroom aerator and aerator manhole provided with a ventilating stack.

The remaining section has a total fall of about 200 ft. The high velocities that this drop allows (from 3 to 10 ft. per sec.) and the frequent drop manholes provided and ample natural ventilation make of this section what is in effect an elongated aeration tank.

A chlorination house installed on the outfall sewer will be used for several months in the summer to supply complete biological protection against pollution of bathing beaches.

The original plans included a pressure section in the outfall sewer, which would have developed a head of 170 ft. An impulse wheel and generator were planned to utilize this drop to generate power to drive motors at the treatment plant. The wheel also would have provided effective aeration. The electric current was to be carried back to the plant by cable laid in the sewer trench. Before contracts were let for the work, however, the local electric power company submitted a special contract at rates that rendered the construction of the small power plant financially inadvisable. Accordingly, this element was omitted from the project. However, the original design still accomplished the desired object of economy in operation.

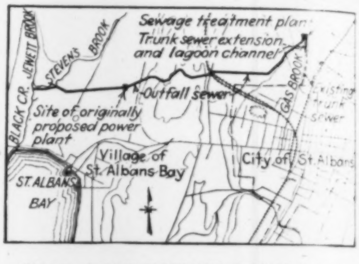


FIG. 3—LONG OUTFALL SEWER detours long bend in stream that formerly received raw sewage, and discharges treatment-plant effluent near head of St. Albans Bay.

Stormwater lagoons

The sewer system of St. Albans operates on the combined principle, but the lateral connections to the trunk sewer are undersized so that the trunk sewer does not receive a full quota of stormwater during wet weather. For two months gagings were taken of the flow of the trunk sewer before the design of the new plant was started, and during that period several storms occurred, including a warm rain that fell on melting snow. The action of the trunk sewer under these conditions was completely charted, and the peak flow was found to be in the neighborhood of 20 cu.ft. per sec.

To take the full wet-weather flow of the trunk sewer from the sewage-works site through the long outfall sewer to St. Albans Bay would have proved an expensive undertaking and would have required a large investment to provide a line, which would only

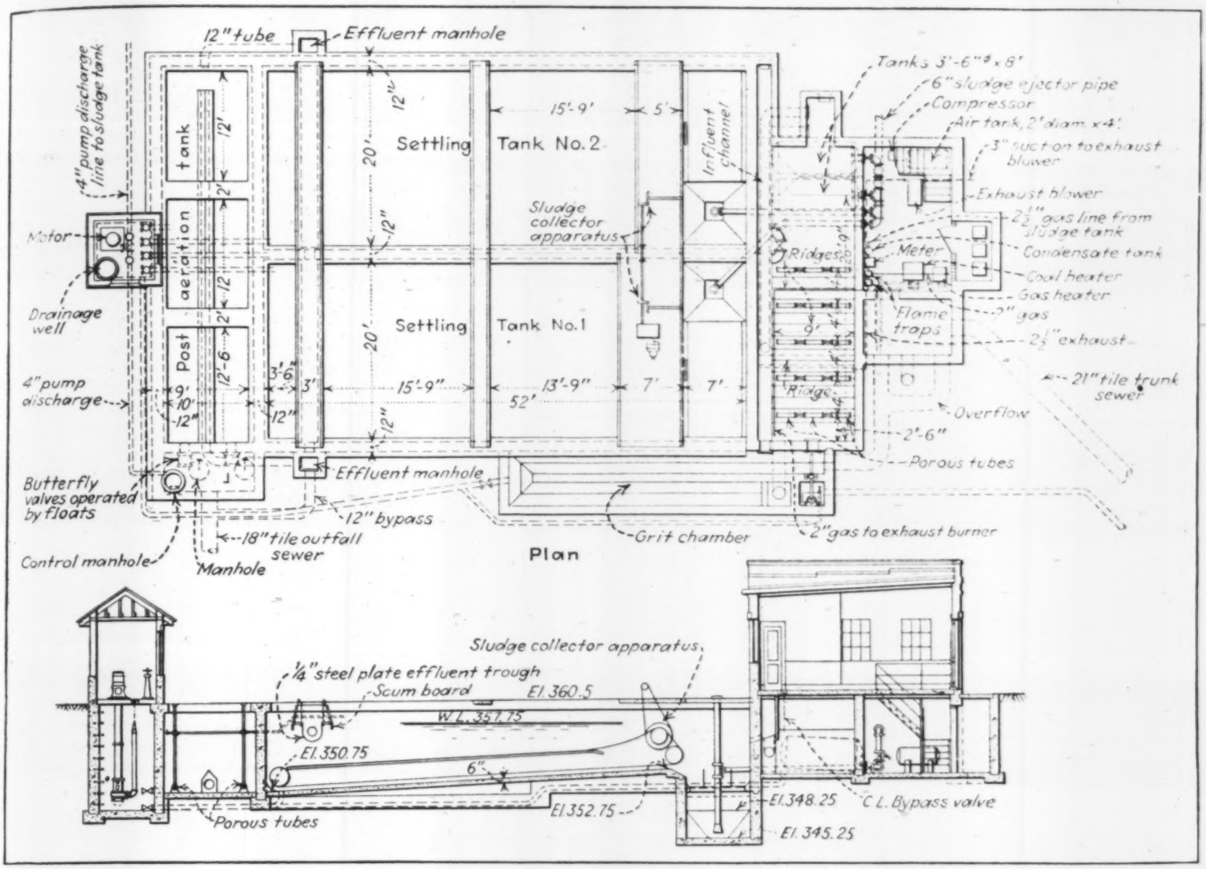

12" tube
Effluent manhole
4" pump discharge line to sludge tank
Post aeration tank
12'
2'
12'-6"
9'
10'
12"
3'-6"
3'
20'
Settling Tank No. 2
Settling Tank No. 1
15'-9'
5'
15'-9"
13'-9"
7'
7'
52'
Influent channel
Sludge collector apparatus
Motor
Drainage well
4" pump discharge
Butterfly valves operated by floats
Control manhole
Manhole
18" tile outfall sewer
12" bypass
Effluent manhole
Grit chamber
Plan
Tanks 3'-6" ⌀ x 8'
6" sludge ejector pipe
Compressor
Air tank, 2' diam. x 4'
3" suction to exhaust blower
Exhaust blower
2½" gas line from sludge tank
Condensate tank
Coal heater
Gas heater
2" gas
2½" exhaust
Meter
Ridges
Flame traps
Ridge
9'
2'-6"
21" tile trunk sewer
Overflow
2'-6"
Porous tubes
2" gas to exhaust burner
Sludge collector apparatus
¼" steel plate effluent trough
Scum board
El. 360.5
W.L. 357.75
El. 350.75
6"
El. 352.75
El. 348.25
C.L. Bypass valve
El. 345.25
Porous tubes

FIG. 4—PLAN AND SECTION of units for new sewage plant at St. Albans, Vt.

occasionally have been utilized to its full capacity. The outfall sewer was, therefore, designed to carry three times the dry-weather flow, or 5.3 cu.ft. per sec. predicted for a future population of 10,000. When the stormwater flow of the trunk sewer exceeds this capacity, the excess will be discharged over an overflow weir and led to storage lagoons at the sewage-works site, which have been built by relief labor. These lagoons will eventually provide capacity for about 2,000,000 gal. and will be used to retain the peak flows until the storm has subsided. After the storm the sewage will return to its normal dry-weather basis of about 1.77 cu.ft. per sec., and there will again be in the outfall sewer a reserve capacity of 3.4 cu.ft. per sec., which can then be used to drain the lagoons. During the gaging of the trunk sewer, it was found that the worst condition would require a lagoon capacity of less than 1,000,000 gal.

The use of the lagoons will be limited to a few storms per year. Any deposit will be harrowed under after draining.

Cost of the improvement

The total construction cost of this sewage project will be about $92,000. The cost of maintenance will be small, and mechanical apparatus has been supplied so that one operator can successfully handle the plant and give part of his time to other duties.

The legal claims for damages which have been experienced in the past will be completely eliminated by taking the sewage or sewage effluent out of the section of the stream where damages have or could possibly obtain in the future. The cost of construction has been held within the limits of the local purse and the taxpayers' consent. The cost of operation will be low and well within the limit of the taxpayers' appreciation. An outfall sewer has been designed to accomplish a double purpose, the conveyance of sewage to a desired point and the continuation of a sewage-treatment process that was initiated at the sewage-works itself.

◆

Survey of Land Settlement Begun by Government at Boulder Dam Reservoir Site

SURVEYS have been begun by the U. S. Coast and Geodetic Survey to determine the deformation of the earth's surface caused by the weight of water to be stored in the reservoir back of Boulder Dam. The total weight of this water is estimated at 41,500,000,000 tons. One survey party will cover an area having a radius of 50 miles from the center of gravity of the reservoir, to establish accurate benchmarks from which to determine vertical deformation; and another party will make an accurate survey of the reservoir area in order to determine whether there are any horizontal movements.

Three types of settlement may take place, according to *The Reclamation Era*: elastic settlement due to compression in the earth's crust, which probably will take place in from two to three years; settlement, in accord with the theory of isostasy, which will take place in an unknown period; and settlement due to plastic flow in the crust of the earth. According to the isostasy theory, the earth's crust floats on a heavier lower material in a plastic state; hence if a weight of appreciable magnitude is placed on the earth's surface at any point there will be a submergence until equilibrium is restored.

Aerodynamics and the Civil Engineer—VII

Wind Velocity in Relation to Height Above Ground

A review of available observations and studies throughout the world—Below the height of the gradient wind at about 1,250 ft., the velocity at 100-ft. height apparently has a value of about 67 per cent of the gradient velocity and increases from 2 to 5 per cent for each 100-ft. increase in height

By W. Watters Pagon
Consulting Engineer, Baltimore, Md.

PREVIOUS ARTICLES in this series have appeared as follows: March 15, 1934, p. 348; July 12, 1934, p. 41; Oct. 11, 1934, p. 456; Dec. 27, 1934, p. 814; April 25, 1935, p. 582; and May 9, 1935, p. 665. The last article discussed the subject of engineering meteorology and thus leads logically into the present article, which is devoted to a phase of meteorology of particular interest and concern to the civil engineer.—EDITOR.

AT THE EARTH'S immediate surface the wind velocity must be zero. The rate of increase with height has been observed by several investigators and has been derived mathematically by others through abstract reasoning and by analogy to similar phenomena in pipes.

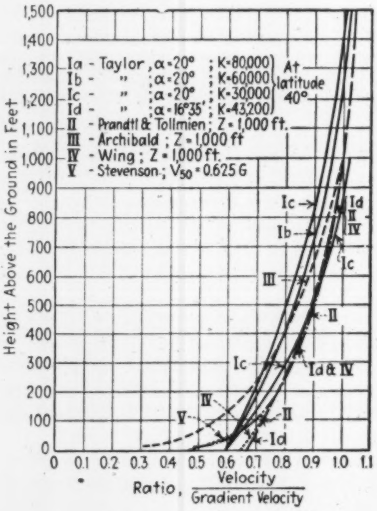
In 1880, Stevenson derived an empirical formula from observations within a height of 50 ft., which may be expressed as $\frac{V}{V_{50}} = \frac{H + 72}{H_{50} + 72}$, where V and H refer to any elevation below 50 ft. For the ground surface, $H = O$, V_o would always have a finite velocity that cannot be true. In 1885, Archibald experimented with anemometers suspended in pairs from kites at elevations as great as 1,300 ft. He evolved a formula, $\frac{V_1}{V_2} = \left(\frac{H_1}{H_2}\right)^{\frac{1}{n}}$ and found the exponent n to vary from 5.2 for instruments at 1,095 and 767 ft., to 2.75 for instruments at 250 and 102 ft., respectively. He recommended an average of 4, but as suggested in the article on eddies (*ENR*, April 25, 1935, p. 582) about 7 or 8 is correct above the laminar sublayer.

These two formulas give relative values only, and require some further data in order to establish the datum. In the accompanying figure, they are plotted upon assumed values of V_{50} and V_2. The former has been plotted as 62.5 per cent of the gradient velocity G, a value computed from data that will be considered later, and V_2 has been chosen for $H_2 = 1,000$ ft. Stevenson's formula when so plotted agrees well with the pipe analogy, but Archibald's formula does not plot well for $n = 4$, nor has any satisfactory device been found to plot the reported values at different relative heights.

In 1915, Wing made anemometer observations on a 492-ft. and a 300-ft. radio tower at Ballybunion, Ireland. He determined simultaneous average velocities at three differing heights in open country 65 ft. above sea level and about ½ mile from the ocean behind low rolling sand dunes 70 ft. high. Calibrated anemometers were mounted on the two towers at about 500 and 300 ft., respectively, and a third one on a 15-ft. post midway, between them, with the weather vane adjacent to it. Continuous readings were made for 10-min. periods, in order to compensate for the tower spacing in gusts and to smooth out the effect of gusts on velocity. It was found that 2 min. sufficed. Six daily 10-min. readings at 12-min. intervals were made on 40 or more days and in winds of 0 to 37 m.p.h. from all directions.

The gradient wind was computed approximately. Wing obtained a formula which gives the relation of velocity at height z, in terms of that near the surface; $v = \frac{1}{2}V_g\left(1 + \sqrt{(1 + 4.77\,z')\,(1 - 0.265z')}\right)$, where $z' = z/1000$. This is plotted in Fig. 1 as ratios V_1/V_2 for $H_2 = 1,000$ ft. (See also *Proceedings*, Am. Soc. C. E., 1932).

CHANGE of wind velocity with height above ground as found by different investigators.



In 1930 Scrase found that velocity varied as the 0.13 power of the height z, for conditions of zero temperature gradient.

In measurements of the boundary layer of bodies and wind-tunnel walls, it has been found that the velocity varies with the $\frac{1}{n}$th power of the distance from the surface. The value of n lies generally between 5 and 7, and the latter is commonly used. Prandtl and Tollmien, comparing wind with turbulent flow in moderately rough pipes with no thermal turbulence, considered n as being 6.38, giving a variation as the 0.157 power of the height z.

In 1931 Bryant analyzed four sets of wind velocities at sea on the basis of Taylor's "eddy conductivity" theory, which is discussed later. He concluded that at sea the largest velocity gradients (i.e., near the surface) occur for low eddy conductivity in the first few thousand feet, which applies to overcast sky and light wind; the lowest gradients occur when there is a clear sky, brilliant sunshine and moderately strong to gusty wind, also in light wind and exceptionally calm sea. His greatest altitude was 130 ft.; hence his work applies to conditions relatively near the surface, but not lower than 15 ft. above it.

The significance of his comments will be clarified below by a study of Taylor's work. Using Taylor's symbols, Bryant's curve for the latter case may be derived from Taylor's formula with $\alpha = 20$ deg. and $K = 1,600$; his curve for "calm sea, overcast sky," approximately with values of 45 deg. and 45,000; and his curve for "average sea conditions, wind speed up to 50 m.p.h."

by the pipe formula. He reports that the Prandtl pipe formula fits well the average conditions for open level country as determined by Simpson, and concludes, "It may well be sufficient for many purposes to adopt this formula for both sea and land conditions."

The eddy conductivity theory

Taylor based his work upon Osborne Reynolds' theorem that the stress dragging the lower air in the direction of velocity increasing is $-\rho\,\overline{u'w'}$, where ρ is the air density, u' and w' the simultaneous deviations at the same point of the x- and z-components of eddy velocity, from the mean velocity. The bar denotes the mean, over a long interval, of the products of the deviations. When air of sufficient velocity moves over a surface of discontinuity, eddies are constantly being formed. Usually the ground forms the surface at which they originate, but for certain conditions of density stratification, generation at the ground may be prevented and may occur at some higher level where stratification ceases. He assumed that the diffusing power of turbulent air is in some way dependent upon the eddy characteristics, and assumed a coefficient K, of "eddy conductivity," similar to the heat conductivity of a solid body, and roughly equal to $\frac{1}{2}\,wd$, where w denotes the average vertical velocity component due to eddies, $\frac{1}{2}$ is a statistical average, and d is the average vertical height through which an eddy rises from the position where it forms to the position at which it has dissipated into the surrounding air, i.e. Prandtl's "mixing length." K may represent the transfer of heat, momentum, or water vapor, or of solid particles in sand storms. He made an interesting analysis of the reason why desert winds in Egypt raise dust to considerable heights in the hot southerly and southeasterly Khamsin winds, but not in the colder winter winds. If the dust particles are of such size as to settle at the rate of 3 in. per sec., and are raised to such heights that the number per unit volume at a height of 1,640 ft. is $\frac{1}{10}$ of the corresponding number at the ground, the value of K is 300,000 in the summer. That turbulence extends to great heights above the earth's surface is demonstrated by the great distances which are traversed by dust raised in western storms of the past two years before it settles back to the surface.

From the equations for *viscous flow* of air on the rotating earth, Taylor derives the flow in two rectangular directions at height z, in relation to the uniform flow of air above the region where the surface conditions and the viscosity affect the flow. This upper "geostrophic" or gradient wind is determined by the barometric gradient and is $G = \dfrac{dp}{dn} / 2\,\rho\omega \sin\lambda$ (*ENR*, May 9, 1935, p. 665). Letting B represent $\sqrt{\omega \sin\lambda / K}$, where K may vary from small values to 300,000 c.g.s. units, and letting α = the angle between the surface wind at $z = 0$ and the gradient wind at $z = Z$, positive if the deviation is clockwise with height increasing, he derives:

$$1 - \frac{u}{G} = e^{-Bz} \sin\alpha\,[\cos(Bz - \alpha) - \sin(Bz - \alpha)]$$

$$\frac{v}{G} = e^{-Bz} \sin\alpha\,[\cos(Bz - \alpha) + \sin(Bz - \alpha)]$$

where u = velocity at height z along the isobars, and v = velocity at height z perpendicular to the isobars. Letting Q be the resultant of u and v:

$$\frac{Q^2}{G^2} = 1-2 \sin\alpha\, e^{-Bz}[\cos(Bz - \alpha) - \sin(Bz - \alpha)] + 2 \sin^2\alpha\, e^{-2Bz}$$

For $z = 0$, these reduce to simple forms, since $e^{-Bz} = 1$;

$$u_o = G[1 - \sin\alpha(\cos\alpha + \sin\alpha)]$$
$$v_o = G \sin\alpha (\cos\alpha - \sin\alpha) \text{ and}$$
$$Q_o = G(\cos\alpha - \sin\alpha)$$

Applying this equation to Wing's results (see accompanying curve), it appears that for $\alpha = 0.2900$ radians and $K = 43{,}000$, approximately, there is good agreement. From $z = 200$ to $z = 1500$, the Prandtl pipe formula agrees well, and Bryant's "average sea conditions" are quite close.

Wing's tests were made at 15 ft. or higher; Taylor's study was based upon slippage at the ground; hence near the ground the pipe formula seems to hold as against the others. Stevenson's formula agrees well when fitted to the curve at height $z = 50$.

Taylor checked his results against tests by Dobson at Salisbury Plain and believes that the close agreement is evidence of the correctness of his assumptions. Dobson found that the gradient velocity is usually attained at about 1000 ft.

In summer, when the adiabatic lapse rate is quite frequently attained, turbulence is readily generated and K may also increase with height. Equating Taylor's equations with the exponential formula, K must vary as the 0.843 power of z. At the surface K depends upon the surface roughness. This would produce more variable speeds near the surface, and might explain the ratio of 0.58 for V_{50}/V_{100} observed at sea on the "Eagle" and the "Scotia." For small values of K the Taylor curve shrinks into the lower right corner of the diagram and for $K = 0$ is coincident with the lines $\alpha = 0$ deg. and $z = 0$; hence, if K is small, the ratio of velocities might be very near unity at all heights except near the ground, where the velocity is dependent only upon the angle α. This would be an unturbulent wind.

In a later paper Taylor reported that while he had found $K = 3{,}000$ approximately for the Grand Banks, the Salisbury Plain tests with pilot balloons indicated $K = 50{,}000$. He then made an approximate determination of K by a study of records taken on the Eiffel Tower in 1890 to 1894 at Els. 59, 402, 645 and 988 ft. His study revealed that over Paris, between 59 and 402 ft., K varied irregularly with the season from 90,000, minimum, to 240,000, maximum. In the upper segment of the tower it ranged from 16,000 in February to 300,000 in July. He concluded that turbulence appears to decrease with height in winter and to increase in summer. In winter the lapse rate up to 1000 ft. is considerably less than adiabatic, thereby tending to prevent or to suppress spontaneous formation of turbulence; in the summer the adiabatic rate is attained in a large number of cases, thereby encouraging the formation of turbulence. Under these circumstances an increase in K with height is to be expected because K is roughly proportional to the vertical component of turbulent velocity and to the diameters of the eddies. It is to be expected that the size of the eddies will increase with height, because they have more room in which to grow.

Since the values of K near the ground have no seasonal characteristic, it is evident that the temperature gradient can have no marked effect near the ground; the turbulence here must therefore be governed by the wind velocity —which shows no seasonal variation— and by the nature of the ground. Within the first 6 ft. this seems to be entirely true.

Taylor analyzed the tests at Salisbury Plain and found that for the first 2,630 ft. the mean values of K for light, medium and strong winds are respectively 30,000, 50,000 and 60,000, increasing with the velocity. His Eiffel Tower results give a range of 100,000 to 200,000 near the ground; and 20,000 to 300,000 near the top, depending upon the season. The mean value for Salisbury Plain is less than that for Paris, as might be expected because of the smoother nature of the surface.

Pressure on buildings

Within the heights where the speed is less than the gradient wind it is of interest to determine the *average velocity* on structures of differing height, by integration of the pipe or exponential formula, since Taylor's formula is difficult to handle. Taylor's equation passes through the gradient velocity to somewhat higher values. The lowest height, Z, at which they are equal may easily be found as

$$\tan\alpha = \frac{\sin BZ - \cos BZ}{\sin BZ + \cos BZ - e^{-BZ}}$$

For $BZ = 45$ deg. $= 0.7854$ rad., $\alpha = 0$; for $BZ = 1.453$ rad., $\alpha = 45$ deg.

Since $B = \sqrt{\dfrac{\omega \sin\lambda}{K}}, = \dfrac{6.86}{1000\sqrt{K}}$ for latitude 40 deg. N, then, expressing Z in ft. (1 ft. = 30.48 cm), $\frac{Z}{\sqrt{K}} = 4.8BZ$.

The following table gives α in terms of $\frac{Z}{\sqrt{K}}$, from which Z can be found for any assumed values of α and K.

BZ Rad.	BZ Deg.	α Deg.
0.7854	45	0
0.8000	45–50	1–13
0.9000	51–30	9–20
1.0000	57–20	16–36
1.2500	71–30	33–05
1.4000	80–10	42–00
1.4515	83–10	44–47

Some equivalent exponential curves have been plotted against curves by Taylor's formula in the accompanying diagram showing a very close agreement when the large uncertainty in the values of K and α is considered. If, following Taylor, we assume for open country $\alpha = 20$ deg. and $K = 60{,}000$ for strong summer winds and 30,000 for winter, and for large cities $\alpha = 45$ deg. and $K = 160{,}000$ and 80,000, respectively, then we can use values of Z from the above tabulation obtaining the following: $\frac{Q}{G} = \left(\frac{z}{Z}\right)^{0.157}$. Since pressure is proportional to (velocity)^2, then, integrating, to find the average (weighted) velocity in height, z, we find $\frac{Q_{av}}{G} = Cz^{0.157}$, where $C = \left(\frac{1}{1.145Z}\right)^{0.157}$.

Table II gives the results for the assumed conditions.

Of course, above height Z, the velocity is considered to be uniform. End flow over the eaves will reduce the average pressure, as computed.

Evidently $Q_{av} = 0.284Gz^{0.157}$ is a fair average for these extreme conditions and eliminates the necessity for consideration of what values of α and K should be adopted. This implies that on the average the gradient wind would be found at 1250 ft. above ground or above some "effective ground level" which Wing properly stresses. It is conservative also, when consideration is given to the fact that gradient wind height in winter is lower than in summer because of more stable stratification, and 1250 ft. is less than either value of Z for cities in the extreme cases chosen. This results in the use of a higher percentage of the gradient velocity.

TABLE I—PROBABLE VELOCITIES Q and Q_{av} IN TERMS OF G

	$z^{0.157}$	Q/G*	Q_{av}/G	Q_{av}/Q
100	2.06	0.67	0.58	0.87
200	2.30	0.75	0.65	"
300	2.44	0.80	0.70	"
400	2.56	0.84	0.73	"
500	2.65	0.87	0.76	"
600	2.73	0.89	0.77	"
700	2.80	0.91	0.79	"
800	2.86	0.93	0.81	"
900	2.91	0.95	0.83	"
1000	2.96	0.97	0.84	"
1100	3.00	0.98	0.85	"
1200	3.04	0.99	0.86	"
1250	3.06	1.00	0.87	"

*See *ENR*, Dec. 27, 1934, p. 814, for variation with height in wind tunnel studies, and note also the diminished pressure near the ground on the front wall of the mill building.

TABLE II—WEIGHTED AVERAGE VELOCITY IN HEIGHT, z, IN PER CENT OF GRADIENT VELOCITY

z, ft.	$z^{0.157}$	Open Country, α = 20 deg. Summer	Open Country, α = 20 deg. Winter	Large Cities, α = 45 deg. Summer	Large Cities, α = 45 deg. Winter
		K = 60,000	= 30,000	= 160,000	80,000
		Z = 1,250	= 885	= 2,020	1,420
		C = 0.284	= 0.300	= 0.265	0.277
100	2.06	59	62	55	57
200	2.30	65	69	61	64
300	2.44	69	73	65	69
400	2.56	73	77	68	72
500	2.65	76	79	70	74
600	2.72	77	81	72	76
700	2.79	79	83	74	78
800	2.86	81	85	76	80
900	2.90	82	87	77	81
1000	2.96	84	89	79	83

To summarize, the weighted average velocity on a building of height H is $\frac{7}{8}$ of the velocity at the top, and the velocity at the top is $Q = G\left(\frac{H}{1250}\right)^{0.157}$

The center of pressure, for buildings of height less than Z, lies at $\frac{1.314}{2.314} =$ 57 per cent of H. If the height exceeds Z, then the center must be modified by including the uniform pressure on the portion above height Z. The end flow over the eaves will slightly lower the center of pressure.

From Table I may be determined the average velocity and therefore pressure on a building of any given height once the gradient velocity and the characteristic quantities K and α are known. If study of these elements can be carried on scientifically in every locality, then a reasonable basis will develop for the establishing of local building-code pressures. Of course the table does not consider the increase in velocity due to gusts, which is discussed elsewhere.

Effect of mountains

The wind blowing up the side or over the top of a mountain can be approximated by considering the mountain as an infinite horizontal half-prism or cylinder. For example, the Blue Ridge Mountains may be represented roughly by a triangle formed by splitting a square prism on a diagonal plane, for which the velocity is given aproximately in *ENR*, March 15, 1934. A single, much-glaciated mountain knob standing alone may be simulated by a hemisphere or by a hemiellipsoid, for which the stream functions are shown in *ENR*, March 15, 1934.

Wind and water gaps present some difficulty beyond that of the mountain range in general. Consider a triangular prism with a saw kerf cut through it parallel to the wind direction. It is clear that there will be much greater velocity through the gap than over the mountain top and that, regardless of the wind direction, the wind through the gap will be almost unidirectional except as modified by local eddies. Approximately, therefore, the wind velocity at the top of a triangular mountain range would be 1.5 times the approach velocity; that at the top of a cylindrical range about as given in the following table for cylinders; that on an isolated mountain of drumlin type would be about as given in the table for spheroids: a/h = base width divided by height:

Elliptical cylinders	a/h	Velocity Ratio
Prolate	4:1	1.25
Prolate	3:1	1.33
Prolate	2:1	1.50
Circular	1:1	2.00
Oblate	1:2	3.00
Oblate	1:3	4.00
Oblate	1:4	5.00

Spheroids	a/h	Velocity Ratio
Prolate	4:1	1.08
Prolate	3:1	1.12
Prolate	2:1	1.21
Sphere	1:1	1.50
Oblate	1:2	2.12
Oblate	1:3	2.74
Oblate	1:4	3.38

Clearly a mountain with abruptly rising sides may produce very high velocities. Of course in the ellipsoidal type there will be high flow around the ends also.

For high velocity on Mt. Washington, see *ENR*, May 10, 1934, p. 604. Velocities such as these are possible in exposed situations where trees and other mountains do not disturb the general flow. For high mountains there must also be considered the general increase in velocity with height and also the change of air density with height, both of which affect the air flow.

Since the density changes with height, two stream lines now enclose a constant weight of air, but not a constant volume. The crowding effect shown in *ENR*, March 15, 1934, will, therefore, not appear as before, but the effect will be the resultant between the narrowing of the tubes of flow and the widening of them due to expansion of the air under diminished atmospheric pressure.

The development of gliding in Germany has caused much study of the rising air currents caused by obstacles, and hence the general nature of flow over and around mountains. There is some level, the "height of influence," at which no deviation of the air flow results, and the height of this above the mountain summit depends upon the shape of the mountain, its height, the lapse rate of temperature in the free air and along the mountain slope, the moisture content of the atmosphere, etc. The velocities found upon a mountain are intimately connected with this height of influence.

Morgan has given an excellent résumé of numerous observations of balloon trajectories, of temperature range, of the path of smoke bomb puffs, etc. For example, von Ficker in a free balloon passed 3280 ft. above the Diablerets, and found his speed increasing from 10.8 to

85 ft. per sec. above the summit and then declining to 42.6 ft. per sec.

If we assume the air to be non-saturated and rising so rapidly that the cooling effect is at the dry adiabatic lapse rate of 5½ deg. F. per 1000 ft., and if the lapse rate in free air at a distance from the mountain is less than adiabatic, then the air rising up the mountain becomes cooler than is the free air at the same height. The air passing across a vertical through the summit then experiences less change at a given height than the free air, until at the height of influence the two become the same. The change above the summit may, therefore, even be inverted, i.e. become warmer with height, according to the difference between the first lapse rate and the lapse rate of the free air. If the free air is saturated, then the saturation adiabatic lapse rate will give an approximate result. On this basis, the height of influence is that height at which the local and free air temperatures become the same. If there is a discontinuity in the free air at some height below the summit, this method is invalid. In the lee of a mountain the descending air may cause clear sky for 5 or 10 miles horizontal distance, even though the mountain is cloud-capped.

Pockels has derived a theoretical result based upon certain assumptions; Ono extended this method to isothermal, adiabatic and pseudoadiabatic cases; Ackeret and Defant studied a half-elliptical profile. Pockels assumed a horizontal, *ideal* flow in two dimensions, neglecting viscosity and surface friction, over a series of identical mountain ranges. By making the surface profile one of the resulting stream lines, he derives a sine-like shape for the ground, with a relatively narrow valley between flat-topped mountains. He finds that the horizontal velocity is maximum at the summit, minimum at the botton of the valley, and equal to that of the free flow at the midpoint of the slope. At this midpoint the vertical velocity is maximum and equal to $u \tan \alpha$, where α is the slope at this point.

Idrac found some confirmation of this theory in observations on a gentle slope (maximum 25 deg.) between a 328-ft. ridge and a plain close to sea level near Biville, but there was a vertical component still at the summit. (See Discontinuous Flow, *ENR*, March 15, 1934.) On ground of similar profile, but of 32.8-ft. height, he found similar results, proportionally. For the windward side he considered his results to be valid from 13.2 to 49.2 ft. per sec. To leeward, he found much turbulence.

Free balloon pilots have found marked eddy formation on the lee side of mountains, sometimes as a stationary eddy, sometimes as a traveling eddy which produces a sinuous flight path slightly inclined upward. On the windward side there may be also a stationary eddy due to colder air flowing down the slope from the summit, but this seemingly disappears at velocities greater than 33 ft. per sec. Georgii believes that below 33 ft. per sec. at the summit and a lapse rate up to the summit less than adiabatic, stationary eddies will form on the windward side, but that above this the turbulence that is developed will prevent this.

In general, there is turbulence on the lee side, and above 33 ft. per sec. stationary eddies seemingly do not form especially for a near-adiabatic lapse rate. There are a few observations of traveling eddies which rise at an angle of perhaps ½ deg. to 4 deg. and persist for "large distances beyond 7 km." (4.35 miles). The path of two balloons approached closely the two coordinate equations: $x = ut + a \sin\frac{2\pi t}{T}$, and $z = wt + a \cos\frac{2\pi t}{T}$, where u and w are the average horizontal and vertical velocities, a the radial distance of the balloon from the eddy center, T the period of the eddy, and t the time. The respective computed values for these two cases are:

	Case A	Case B
u =	16 mph	11.4 mph
w =	1.1 mph	0.1 mph
Inclination =	3°51′	0°44′
Period =	7′47″	7′45″
Diameter = $2a$ =	460 ft.	290 ft.
Horizontal wave length =	2.1 miles	1.5 miles
Vertical wave length =	740 ft.	100 ft.

The wave lengths given are those of the balloon.

Wind along sea coasts

When the wind passes from sea to land, even though the land be roughly at sea level, there is a marked increase in the surface friction. The consequent retardation of the lower air causes a thickening of the flow sheets, in effect simulating a small hill. This results in increased velocities in the upper air. For offshore winds these statements are reversed, and down-currents might be found even at a height of 1500 ft. There are also temperature changes. Peppler reports the following ascent:

Height, Ft.	Temperature, Deg. F.	Wind Direction	Wind Velocity, Ft. Per Sec.
0	52.2	SW	36.2
658	49.4	SW	72.2-82.2
1640	49.6	W	85.6-95.6
3280	47.2	W	72.2

This shows a slight inversion between 658 and 1640 ft., and the upper limit of up-currents is to be looked for at about 1640 ft.

Effect of trees and buildings

Georgii reports a soaring flight at Cranz "where turbulent motion could only originate either from buildings or from trees." Soaring was possible at 230 ft. There was no difference in the ascending wind in passing from the region of buildings to that of trees. There were no dunes to cause vertical velocity, but dunes may add measurably to the coastal effect.

If the atmosphere is stable, balloons have been found to follow closely the ground profile. But with superadiabatic gradients, air masses may suddenly shoot up to great heights from no discernible cause. A balloon in the Rhone Valley followed the ground profile for 8 min. during a superadiabatic condition, and then rose suddenly to 2300 ft. at a vertical velocity of 6.56 ft. per sec. Even in flat country sudden upward surges may be encountered with velocities of 16 ft. per sec. Under cumulus clouds vertical velocities up to 40 ft. per sec. have been measured.

Timber Versus Steel for Temporary Tunnel Support

ON THE Mono Craters water tunnel now under construction by the city of Los Angeles, after trying out both timber and steel as temporary supports in the tunnel, the conclusion was reached that steel is preferable in this case.

Steel sets can be assembled in about half the time required for wooden plumb posts and a five-segment arch. Steel supports make possible a somewhat smaller tunnel section, with a corresponding decrease in cost, construction time and materials used. After the steel ribs are embedded in the concrete lining, they become an advantage in the form of reinforcing, whereas timber may become a menace to concrete in which it is embedded.

Timber supports dry out in the constantly moving air of the tunnel. Although they are often wet thoroughly just before pouring the lining, swelling of the timbers has been known to crack the lining, particularly where the overlying thickness of concrete is not great. In the tunnel in question specifications call for only 4 in. of concrete inside the temporary supports; the section is a horseshoe, 9 ft. wide inside the concrete.

A disadvantage of the steel sets is that where there is a considerable amount of overbreak the steel ribs are not quite as convenient for supporting the back packing as are the larger timber sets. Another argument advanced by old tunnel men in favor of timber is that timber sets have a "groaning point," and long before they break under a steady load they give warning. On the Mono Craters tunnel, however, the loading thus far has been very light.

Timber sets in the Mono Craters tunnel are 8x8 in. in section and are spaced on 5-ft. centers. Steel ribs for comparable service and spacing are 5-in. I-beams weighing 12½ lb. per lin.-ft. These ribs come into the tunnel in two sections, arranged to splice at the crown. Although timber lagging is used with the steel sets, the plan, to insure complete embedment of the steel ribs in the concrete lining, is to place small concrete blocks behind the steel ribs, thus keeping the timber lagging away from the steel and permitting the concrete to flow entirely around it.

Load Transfer at Transverse Joints an Economic Necessity

Rational demonstration that effective shear joints enable practical saving in slab thickness for a given load or enable heavier loading for a given thickness

By Clifford Older
Consulting Engineer, Wilmette, Ill.

EFFECTIVE load transfer at transverse joints is an economic necessity in concrete pavements because the slab on the far side of a free joint must be strong enough to support about three times as much load as it would in case a joint is used that provides effective load transfer by shear alone. The truth of this statement is developed in the following discussion.

In considering the effect of wheel loads as they approach and pass transverse joints, wheel loads of unity might be assumed if desired. However, as the effect of definite wheel loads of a common magnitude is more easily visualized, wheel loads of 8,000 lb. will be assumed for the time being.

Development of theory

Case 1—If the transverse joint is so constructed as to offer effective shear resistance, both slabs must deflect alike as a wheel load approaches and passes the joint (Fig. 1a). Then, if the adjacent slabs are of the same stiffness and the subgrade support on both sides of the joint is alike (conditions that may be assumed in this case), the ends of the two slabs will share the wheel-load burden equally. This is true because the resistance of an elastic body to bending is in proportion to deflection, and the deflections in this case must be equal. The character of the subgrade support is immaterial providing it is uniform.

In the case of the 8,000-lb. wheel load, the end of each slab needs to be strong enough to support only one-half the load, or 4,000 lb. It may easily be seen that this is true whether the critical fiber stress is under the load and parallel to the joint at the bottom of the slab due to positive moment, or is at the top of the slab some distance from the load and at right angles to the joint due to negative moment.

The deflection at the joint in this case may be called 1*d*. If the subgrade is plastic or only partly elastic, there may be a residual depression of the subgrade in the neighborhood of the joint as indicated in Fig. 1b. This residual depression may be called 1*s*.

The subgrade may actually be rutted the entire length of the pavement under the wheel path. If so, the rut would be deepest at the joints because of the greater deflection of the slabs under load at these points.

Case 2—When the wheel load approaches a free joint, the conditions are quite different. In Fig. 1c the wheel load, moving from right to left, is assumed to be as near the joint as possible without any overlap of the tire. It is obvious that, at this moment, the near slab *N* and the subgrade thereunder must support the entire wheel load. Whether the subgrade is elastic or plastic and whether the critical tensile stress is that due either to the positive or to the negative moment, the maximum tensile stress will be twice as great as in Case 1, providing the slabs are of the same thickness and the subgrade reaction is proportional to subgrade depression. The deflection of the slab will be 2*d*, and the residual depression of the subgrade 2*s*.

Case 3—When the rapidly moving wheel load passes the free joint, the conditions are again different. As the wheel passes the joint, the load is quite suddenly applied to the far (or *F*) slab. At 40 miles per hour the whole load passes from the *N* to the *F* slab in about 1/60 sec. It is a known law of mechanics that a load suddenly applied, in the theoretical sense, is equivalent to twice the static load. V. L. Glover, engineer of materials, Illinois highway department, reports a factor of about 1.5 instead of 2 and calls attention to the fact that if one side of a free joint is at a different elevation than the other, either a drop impact or a combination of raise impact and sudden load occurs. (Illinois Division of Highways, "Reports of Investigations," Vol. 1, 1931.)

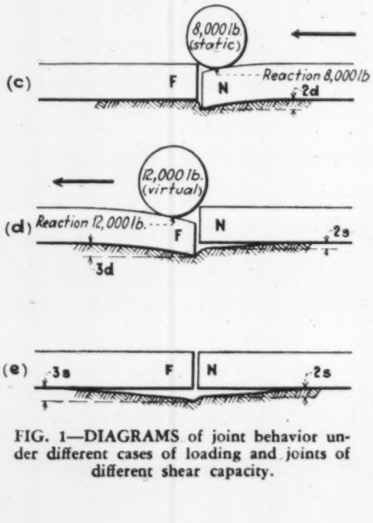


FIG. 1—DIAGRAMS of joint behavior under different cases of loading and joints of different shear capacity.

We are then faced with the condition illustrated by Fig. 1d. We cannot escape the fact that with wheel loads of 8,000 lb. moving at modern speeds, the slab *F* at a free joint must support a virtual load of 12,000 lb., or perhaps considerably more.

In the case of the slab *F* the tensile stress will then be three times as great as in Case 1. The deflection of the slab and the residual depression of the subgrade will be three times as great (3d and 3s), providing again that the slabs are of the same thickness and the subgrade follows the same law. The final residual depressions of the subgrade in the case of free joints may be illustrated by Fig. 1e.

Conclusion—Can there be any doubt that joints that provide effective shear greatly increase the supporting capacity of a pavement of given thickness? Also, if a pavement with free joints is thick enough to carry a given load without structural failure, a thinner and therefore cheaper pavement with effective joints will carry the same load. Neglect to take advantage of one or the other of these two alternates is hard to understand.

Effect of subgrade support

If all subgrade materials were perfectly elastic at all seasons of the year and the elastic properties were known, there would be no residual depression under the joints or elsewhere, and design methods based upon the elastic properties of the materials involved could be used with confidence. Unfortunately, there is no doubt that the actual reaction of subgrade soils and their displacement under load vary in all sorts of ways. They may vary from place to place, depending upon the character of the soil, and from time to time, depending upon changing moisture content, freezing, thawing, compaction under load, settlement, etc. Granular soils may be stable and elastic under the loads imposed, or they may shift or compact more where the deflection of the slab is greatest. Clay soils may be elastic at one time and plastic or even semi-liquid at another, and their behavior may depend upon the intensity of pressure and the area over which it is applied.

That depressions at free joints frequently exist is evidenced by the pumping of mud and water that has often been observed as heavy loads passed. The fact that the end of the far (F) slab is often found depressed and the fact that on heavy-traffic roads transverse breaks parallel to the joint and a few feet away are not infrequent are further evidences. The seriousness of this condition may be illustrated by making plausible assumptions and then computing stresses by simple means.

Assume that subgrade support is virtually absent for a certain distance on each side of a free joint, so that the ends of the adjacent slabs act as practically unsupported cantilevers. Assume that the center of the load application is 6 in. from the end of the slab just before the tire begins to overlap the joint, and that an axle load of 16,000 lb. is uniformly distributed over the entire width of a 10-ft. lane. The load would be 133 lb. per inch of width, and if the end of the slab is unsupported for a distance of 2½ ft., the moment arm would be 24 in., and the bending moment 2½ ft. from the joint would be 3,200 in.-lb. The application of the simple beam formula, $S = 6M/t^2$, shows that the fiber stress in a 7-in. slab would be about 400 lb. per sq.in. If the sudden application of the wheel load to the far slab F increases the effect of the wheel load 50 per cent (as found by Glover), the fiber stress would be increased to about 600 lb. per sq.in. If the unsupported distance should be 4 ft. instead of 2½ ft., the above stresses would become about 700 and 1,000 lb. per sq.in., and structural failure would be certain. The above are by no means unreasonable assumptions.

If the joint is so constructed as to cause both slabs to deflect the same amount, there is no sudden application of the load to either slab, the residual depression of the subgrade is decreased, and the possible cantilever distance may be shortened. Thus, the maximum stress that would otherwise apply is not only reduced by 66 per cent but is still further reduced by virtue of the fact that the moment arm may be reduced.

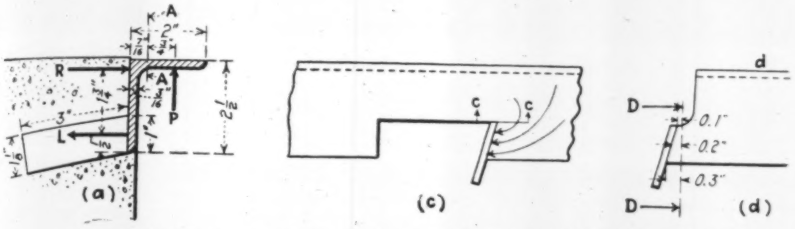


FIG. 2—SKETCHES for analysis of stresses in metal parts of steel air-cushion expansion joint (Fig. 4a).

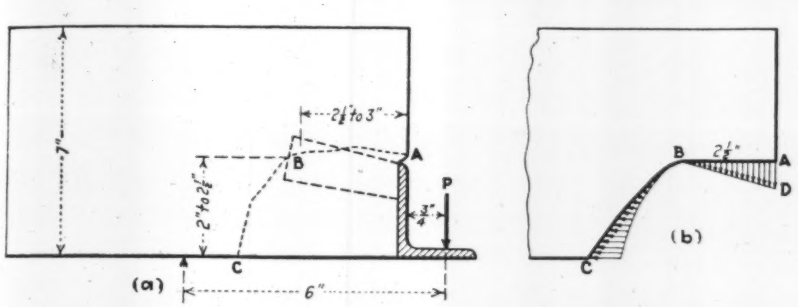


FIG. 3—STUDY of concrete stresses in steel air-cushion expansion joint (Fig. 4a).

Even if we should disregard all theory, we would still be forced to conclude that free or imperfectly doweled joints and cracks seriously unbalance the design, as it is adjacent to them that structural failure, sufficient to require expensive patching, is always found.

Dowel-bar joints

The ideal joint would be one that would be watertight and capable of offering not only adequate shear resistance but also the same bending resistance as that of an unbroken slab.

There are difficulties that at the present time seem unsurmountable in attempting the design of such a joint. For example, the bending resistance per foot of width of a 6-in. slab (ultimate) is about the same as that of a 1.8-in. square steel bar. Again, it would require a continuous steel dowel plate about 0.8 in. thick to offer the same bending resistance at fiber stress of 30,000 lb. per sq.in. as a 7-in. concrete slab at 400 lb. per sq.in. In addition, separate dowel bars, even if absolutely parallel to the direction of motion, introduce great difficulties in the way of concentrated bearing and shear or diagonal tensile stresses in the concrete.

It does not seem that the use of dowel bars is likely to afford the economical solution of the problem.

Design of a shear joint

Every joint used in concrete road work should be sufficiently rugged so that it will not be injured by ordinary handling, by vibrating or by a vigorous spading and tamping of the adjacent concrete. It must be able to accomplish load transfer without involving stresses in either steel or concrete in excess of the usual working stresses, and it should not be unduly costly.

The structural-steel expansion joint shown by Fig. 4a and the contraction joint shown by Fig. 4b effect load transfer by shear only. When the concrete has taken initial set, the temporary bolts at the top of the expansion joint are taken out and the concrete above the bearing plate is removed. The contraction joints need no attention after the concrete is poured.

The design is such that when the concrete hardens, the end of one slab extends between, and makes a tight fit with, the projecting horizontal legs of the pair of continuous steel angles that are rigidly anchored to the end of the adjacent slab. There is, therefore, no lost motion. Hence, as soon as one slab starts to deflect under load, the other must deflect a like amount.

Calculated strengths

The strength of the expansion joint may be tested as follows: When a wheel load approaches this joint, as in Fig. 1a, the resistance of slab F to deflection produces an upward reaction on the horizontal leg of the upper angle. When the wheel load has passed the joint, the end of slab F produces a downward load on the horizontal leg of the lower angle. As the reactions are the same in either case, only one angle need be analyzed. An axle load of 18,000 lb. will now be assumed. As each slab offers the same resistance to deflection, the horizontal leg of the angle is required to support half of the total load, or 9,000 lb. Assuming that 7½ ft. of the joint (less than the width of a heavy truck) transmits the entire 9,000 lb., the load per inch of joint may then be taken as 100 lb.

Bending Stress in Angle—Refer to Fig. 2a. With the joint fully open (a condition that prevails only when the pavement is new), the 100-lb. load per inch, P, is applied 0.75 in. from the critical section A-A, and the bending moment at this section is 0.75 P, or 75 in.-lb. The formula $S = 6M/t^2$ applies. The thickness being $\frac{3}{16}$ in., a fiber stress of about 13,000 lb. per sq.in. is indicated. An ample margin of safety in bending is obvious.

Tension of Lugs—Refer again to Fig. 2a. As the lugs are spaced 6 in. c. to c., the load per lug is 600 lb. The arm of the resisting couple R-L is taken as $\frac{1}{4}$ in. less than the distance from the center of the lug to the top of the angle. Taking moments about R, the tension on each lug at the base of the lug is then $600\times1.1875/1.75=407$ lb. The area in tension at the base of the lug is $\frac{3}{16}$ sq.in. The unit tensile stress is therefore about 2,200 lb. per sq.in. This is of course but a nominal stress.

Lug Bond—The lugs are wider at the outer than at the inner ends, so that the bond will be partly mechanical. The surface area of the sides plus the top and bottom edges is 7+ sq.in. The 407-lb. load distributed over 7 sq.in. results in a unit bond stress of less than 60 lb. per sq.in. The bond stress is therefore not critical.

Shear on Lugs—The vertical reaction per lug (600 lb.) divided by the area in shear ($\frac{3}{16}$ sq.in.) results in a unit shear of 3,200 lb. per sq.in., which is low.

The horizontal shear (Fig. 2c, section C-C) at the junction of the lug with the vertical leg of the angle cannot be computed accurately. Only a part of the tension on the lug is transmitted by shear. Part is transmitted by the bending resistance of that portion of the vertical leg that is below the top of the lug. This bending resistance is roughly illustrated by the arrows.

That there is no danger of horizontal shear failure may be shown as follows: assume (Fig. 2c) that, beginning at the top of the lug, the metal of the vertical leg is sheared to the right 0.1 in. and then, curving upward, the entire angle is cut through. (Fig. 2d results. The tension on the lug (previously computed) is 407 lb. The bending moment on section D-D would be 407 times 0.2, or about 81 in.-lb. The fiber stress would be six times $81/(\frac{3}{16})^2$, or about 14,000 lb. per sq.in. As the bending resistance alone is sufficient, this indicates that the shear at the top of the lug is not critical. In addition, tests to destruction show no shear failure at the tops of the lugs.

Concrete Bearing Stress—At the top of the joint the bearing load is transmitted to the F slab through the bearing plate. The unit bearing pressure is therefore less than at the bottom of the joint, where there is no bearing plate attached to the slab. The width of bearing of the F slab on the horizontal leg of the bottom angle is about $\frac{3}{4}$ in. The load per inch of joint being 100 lb., the bearing stress would be only 400 lb. per sq.in., if the bearing area were only $\frac{1}{4}$ in. wide. Obviously, the bearing stress is not critical.

Tension in Concrete — Definite analytical methods are not available. In order to check the possibility of failure at a corner, as outlined by the line A-B-C (Fig. 3a), samples were made up and tested to destruction by applying a load at P. It was found that when tests were made at 28 days, there were no signs of tension failure in the concrete. Ultimate failure occurred when the horizontal leg of the angle bent downward under a load of about 400 lb. per inch of joint.

In tests made when the concrete was five to seven days old (considerably less than the usual curing period), a continuous strip of concrete pulled off along the corner. This strip had the average dimensions shown in Fig. 3. Tests at ages less than the usual curing period are, of course, of no significance except as they indicate the character of possible failure. The fact that at early ages a continuous strip was pulled off indicates clearly that load transfer from angle to concrete is not unduly concentrated at the lugs. No doubt a substantial portion of the load is transmitted to the concrete by standing friction because of the pressure at the reaction R.

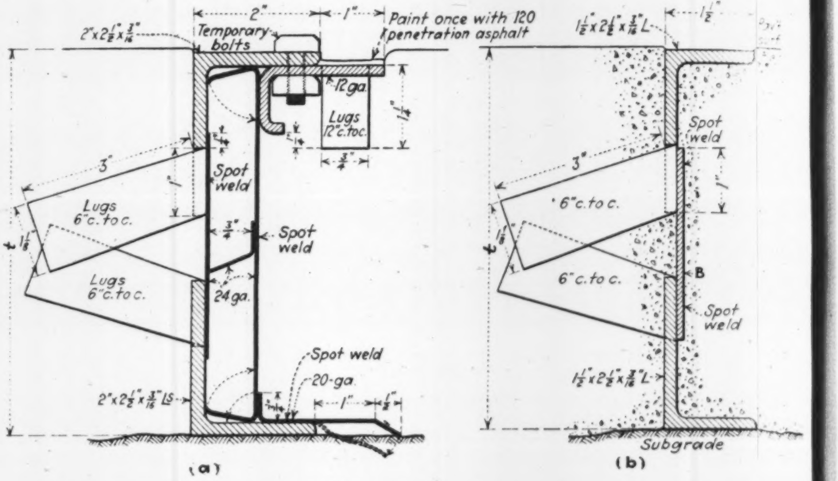


FIG. 4—DESIGN OF SHEAR JOINTS for expansion and contraction in concrete pavements.

Although definite analytical methods are not available for such a corner, an analytical test may be made. The conditions may be represented as in Fig. 3b. At the ultimate load that caused bending failure in the leg of the angle the vertical load P was about 400 lb. per inch of joint. The average tensile stress on section A-B would then be $400/2.5$, or 160 lb. per sq.in. The maximum fiber stress A-D would be twice the average, or 320 lb. per sq.in. If we should assume that the load is all concentrated at the lugs (contrary to the indications of the tests at early ages), A-D would be equivalent to the height of a pyramid of stresses and would then be three times the average, or about 500 lb. per sq.in. This still being under the modulus of rupture, no tension failure would be expected, which expectation was borne out by the 28-day tests.

Because of the fact that the angles are rigidly bonded to the edge of the slab, these angles function in exactly the same way as would marginal reinforcing bars embedded in the top and bottom corners of the slab. They add at least as much strength as would a $\frac{5}{8}$-in. round bar in each of the four corners adjacent to the joint. The edge stress (bottom) and the corner stress (top) are thus still further minimized.

Contraction joint

The foregoing analysis deals with the expansion joint shown by Fig. 4a. The corresponding contraction joint shown by Fig. 4b is extremely simple in design, as no arrangement for initial expansion space is necessary. As it accomplishes "load transfer" in exactly the same way as the expansion joint and provides the same thickness of metal, lug dimensions, etc., the factor of safety is much greater, as the moment arm of the load P is much shorter.

Both analysis and load tests show that joints may readily be designed for shear and, for loads of 9,000 lb. per wheel, may easily have a factor of safety of 4 or more. The usual road slab does not often have as large a factor of safety as this.

Unpublished load tests of such joints in actual service show them to have outstanding efficiency in load transfer. This is no doubt due to the stiffness of the horizontal legs of the continuous angles, which bend but slightly under normal loads, to the absence of lost motion and to the low bearing pressure that eliminates spalling at the bearing edge.

It is clearly indicated that the use of efficient load-transfer joints permits a substantial reduction of slab thickness for given loads, or will greatly increase the load-supporting capacity if the slab thickness is not reduced.

The cost of maintaining properly designed and properly installed metal joints is zero, and this alone offsets the additional capital charges.

AN ENGINEER'S RECOLLECTIONS

By JOHN F. STEVENS
*Consulting Engineer,
Baltimore, Md.*

·VI·

Railway Reconnaissance Over the Cascade Range

A pass through the mountains located, only to find the west slope too steep for a railway line—Inspiration leads to a solution of the west slope problem

The Following Article is the sixth of a series of recollections of engineering experiences recorded by John F. Stevens. The earlier articles were published as follows: March 21, p. 412; March 28, p. 450; April 11, p. 521; April 25, p. 590; and May 9, p. 672. —Editor.

After I had submitted my report upon Marias Pass to the chief engineer of the Puget Sound Extension of the Great Northern Ry. in December, 1889, I was idle during the remainder of that winter. Early in the spring of 1890 I was given the task of finding the best line from Spokane to Puget Sound, a territory of 33,000 square miles.

The problem as to where and how the line could best be gotten out of Spokane was easy to solve. But beyond the center of the Big Bend country I faced a wholly different proposition. While some cursory reconnaissances had been made, I was told that nothing of value had been developed, but I never saw any reports. The problem remained to be solved.

Controlling factors

The selection of a crossing of the Columbia River was a serious matter, as that mighty river, skirting the plateau, lay from 700 to 900 ft. below it, and so close to the plateau that only in a few places could the climb be made except on foot. But the location of the crossing of the river depended upon a larger factor, one that was the key of the whole situation: where could, or should the Cascade Range be crossed? This was a question that could only be correctly answered after a thorough exploration had been made of the whole range, from the British Columbia line south to Stampede Pass, through which the Northern Pacific Ry. finds its way. As a decision had to be made and preliminary surveys be gotten under way that season, it was a job of considerable magnitude.

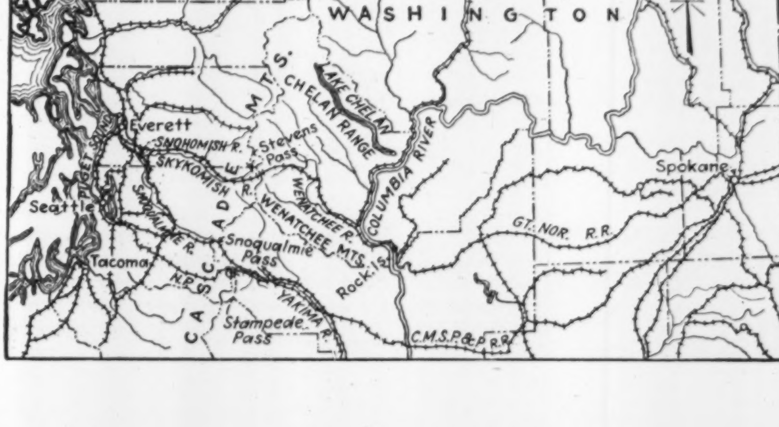
I personally examined every nook and corner of that area which seemed to offer any prospect that might be worth consideration. C. B. Haskell, an engineer of whom I have spoken in a previous article, was my only assistant that summer. He was a fine man and a competent engineer, but, naturally, I preferred to make my decisions on the key matters from personal knowledge, only using Mr. Haskell to fill in details. He gave excellent service, and it was a tragedy that the next year he was drowned by the upsetting of a canoe on the Columbia River.

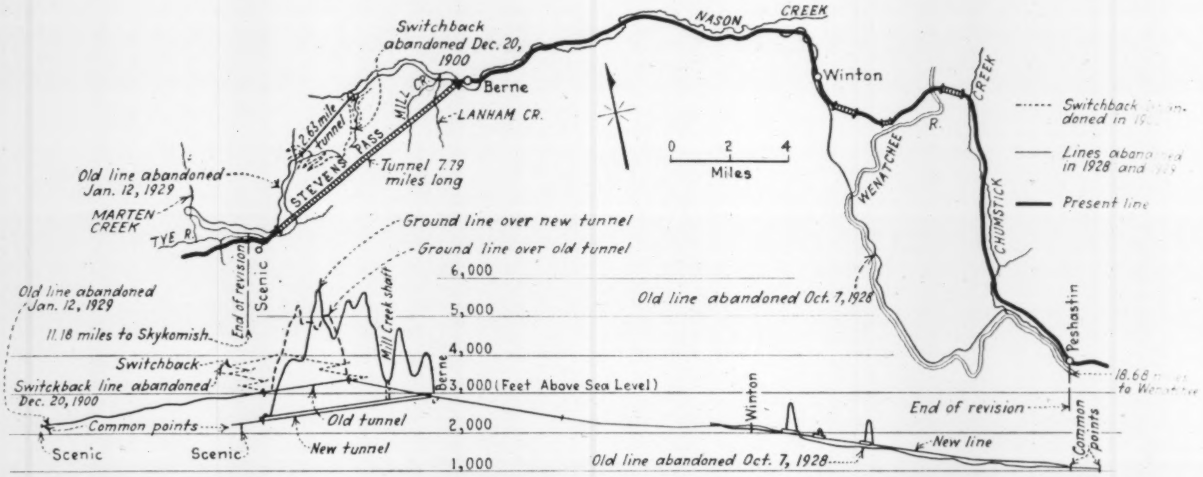
The cross-section of the results of all these examinations proved conclusively that somewhere in and through the range drained on the east by the Wenatchee and on the west by the Snohomish rivers a crossing must be found; but where? The Cascade Range is made up of an almost constant succession of craters of small, long-extinct volcanoes, entirely covered with heavy forests, especially on its western slope. Its western escarpment is very abrupt, falling in places 5,000 ft. in 15 miles. And its narrowest place, its waist, so to describe it, is between the various headwaters of the above-mentioned rivers. Logically, therefore, somewhere at the head of these two rivers there must be a practicable pass, if one existed anywhere.

Lost to the profession!

Some great engineers were lost to the profession when a lot of men wasted their time in trapping and prospecting. While I was busy on reconnaissance in the Cascades one of these embryo engineers wrote to Jas. J. Hill that I was all wrong in my selection of a pass, that there was a much better one that I had overlooked, sending him a sketch and all about it. The chief sent me the letter and sketch with orders to investigate it at once. I knew of course that no such a pass existed in the main range, but orders were orders, and so I went personally to explore the alleged pass. I found the prospector's old camp and went up the creek as shown on the sketch. The search itself took only one day, and I camped that night exactly where I did the night before, without having to retrace my steps. His pass was not in the main range at all, but

THE GREAT NORTHERN RY. gave to Mr. Stevens the task of determining the route that it should follow from Spokane to Puget Sound. As a controlling factor, he first located a pass through the Cascade Range. It was found in what is now known as Stevens Pass.




Switchback abandoned Dec. 20, 1900
Berne
LANHAM CR.
MILL CR.
2.63 mile tunnel
STEVENS PASS
Tunnel 7.79 miles long
Old line abandoned Jan. 12, 1929
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Old line abandoned Oct. 7, 1928

A SWITCHBACK LINE was first used by the Great Northern Ry. to cross the Cascade Range; but its route through Stevens Pass was chosen only after Mr. Stevens had determined that the line and grade could be improved subsequently by the construction of tunnels.

through a side spur of it. The net result of the information—which was to be rewarded—was that I wasted three days that I could ill spare at that particular time.

On one of my cruises up the Wenatchee River I had noted a large creek coming into it from the south, well up into the mountains. I knew that its course could not long continue from the south, and so followed it up a short distance to where it turned abruptly, coming from the west. Realizing that it must head at a summit in the main range, I filed that mental picture away in my head for future investigation.

"Stevens Pass"

A short time afterward when I was cruising the top of the main range, I found a comparatively low place in it, against which a creek flowing east had its head. I felt certain that it must be the beginning of the same creek that I have noted above. Marking this gap, I continued on my way along the range for some days, but on my return from this cruise I immediately sent Mr. Haskell to go up the Wenatchee River to the mouth of the creek, and on up it to its head, which I was confident would be found at the gap I had marked by blazing trees. And so it proved. Mr. Haskell blazed a cedar tree and marked it "Stevens Pass." There was no visible evidence that the pass had ever been visited before, but it probably had been by some wandering woodsmen or prospectors, and the above is the story of how the pass got its name.

Favorably for a railway line

I visited the pass immediately and made a close study of it and of its approaches, and decided that my quest was ended. Geographically, it lay in a favorable place, was comparatively low in elevation (4,000 ft.), afforded easy slopes for the construction of a switchback line and opportunity for a reasonably short tunnel whenever the traffic demanded the latter.

Such a tunnel was completed while I was chief engineer, seven years after the original construction of the line, with a summit elevation of 3,348 ft. That tunnel was 2½ miles in length. It was built entirely by company forces, under the supervision of Harry W. Edwards, resident engineer, and Melville E. Reed, assistant engineer. The headings met almost exactly at equal distances from the portals, with remarkable results. Errors: alignment, ¼ in.; grade, ½ in.; distance (outside measurement carried over two high mountains), 1½ in. I know of no other case under similar conditions where this record has ever been equaled. Incidentally, I may remark that on Aug. 11, 1925, I made a report to Ralph Budd, president of the Great Northern Ry., upon the feasibility of a long tunnel through the pass. My report was adopted, and the nearly 8-mile-long tunnel was put under construction the following October, completed and went into operation in three years—a fine record.

During the summer of 1925 the Stevens Pass Highway was opened with ceremonies. Where years ago we had to struggle up the steep slopes with our packs on our backs, tearing our weary way through the tangled forests, over rocks and through swamps, one can now roll luxuriously in a Pullman or an automobile and give not a thought to, or care in the least how the miracle was wrought by the pioneer engineer.

The west slope problem

Although I felt well satisfied that the pass which we had developed was the one I had been hunting, I realized that a very difficult problem lay on the west side of the range in finding a practicable line, one within reasonable limits of standards and cost, to enable the descent to be made from the tunnel which would be built to eliminate the switchback and lower the grade down to Puget Sound. The season was getting late, but I hurriedly organized a party of engineers and set it at work running a preliminary line down the west side from the portal of a prospective tunnel to the easy valley below. This work continued until deep snow on the mountain slopes made it impracticable to proceed farther. When I got the results of the survey, I was bitterly disappointed and felt that my hard season's work had gone for naught, and that some one better qualified than I had better undertake to solve the problem.

A solution found

For three months that winter I thought of little else than of that part of the Cascade Mountains down which the engineers had pushed a line, which I well knew was impracticable, from its length, tortuous character and prohibitive cost to construct. But somehow I felt that *the* line that I wanted was there, if only it could be developed. Finally one night late in March an inspiration came to me—sometimes I think that it was a dream—and at daybreak I started for the mountains, picking up my hardy and faithful packer, John Maloney on the way. Stopping at the last place where shelter could be found, an abandoned lumberman's shack, we pushed on in the morning over the snow while it would bear our weight.

Finally, reaching a small side valley which I had mentally visualized two nights before, I sketched on paper what I had imagined or dreamed. And this sketch, made under those conditions, translated first into a line of stakes and later into two parallel lines of steel, made certain the rather famous Marten Creek loop on the Great Northern Ry., which made a possible line through a topographically very rugged country.

Location and construction

With the coming of the spring of 1891 and the gradual disappearance of the snow, I organized a sufficient number of parties of engineers, and the work of making the final location of the line over the route through the mountains which I had selected was prosecuted with vigor. My time was wholly taken up with the field work, being spent almost entirely with the engineers.

As soon as all the necessary location engineering was completed, the line was put under construction, and I was given charge of it from the summit to Puget Sound. The work was completed and the track laid over it, mostly from the west, connecting with that laid from the east, in October, 1893, at a point 12 miles west, by rail, from the west portal of the old tunnel. It was here that the last spike was driven in the line of rails which Jas. J. Hill's genius and untiring energy had stretched across prairie, plain and mountain, nearly 1,800 miles, from St. Paul to the Pacific Ocean.

Columbia River crossing

To refer again to the crossing of the Columbia River: once the valley of the Wenatchee River was decided upon as the route, then a crossing of the Columbia River, as near the mouth of the valley as gradient and alignment would permit, was inevitable. It was chosen a short distance above Rock Island Rapids, at a narrow point on the river. I had nothing to do with this part of the line in 1891. If I had been in charge I would have located the bridge some 1½ miles farther upstream, resulting in a longer bridge, but not more expensive, with a shortening of the line and a decided decrease in the total curvature and in the rate of curvature.

Mr. Haskell and a couple of packers comprised my entire staff while I was employed on reconnaissance during the season of 1890. It was a busy season for me, and a most satisfactory one, measured by the results obtained. It called for hundreds of miles of travel, on horseback, on foot and by rowboats. When it came to packing on the back, there was no distinction between the chief and the hired packer. Hazardous situations were encountered, but determination and good luck carried me through without serious mishap, and I felt that what that summer had developed, leading to rapid advancement in my profession, more than compensated for the hard work done.

Made chief engineer

As soon as the track was laid and the line was being made ready for operation, all engineering matters connected with the extension were placed in charge of N. D. Miller, chief engineer of the system, with headquarters in St. Paul. I was appointed assistant chief engineer with office at Spokane, my territory covering the lines west of Minot, N. D., to Spokane, and took up my new duties, which were varied and severe. On Jan. 1, 1895, I was appointed chief engineer of the Great Northern System, succeeding Mr. Miller, and moved to St. Paul. I held the office for eight years before I resigned to go to the Rock Island R.R., first as chief engineer and later as vice-president.

It was the coming of the rails of the Great Northern Ry. in 1893 which first put Seattle on the map and gave it a start to become the commanding banking and commercial center and ocean port of Puget Sound. A practical illustration of this fact is the quickening effect of the low rates on lumber and shingles, which Mr. Hill at once established. The building of the Great Northern docks, elevator and export flour warehouse, together with its trans-Pacific line of ships, made the city an important port. Business got such a start that when the Alaskan boom arrived its wide-awake citizens secured the major part of the Alaskan trade. Finally the Northern Pacific Ry., which had been ignoring Seattle, saw the handwriting on the wall and built a direct line into the city, which has become the real Puget Sound terminus of that road. All this development and growth of Seattle were the direct results of the forward movement caused by the Great Northern's extension to Puget Sound, and it owes its awakening from somnolence to that railroad.

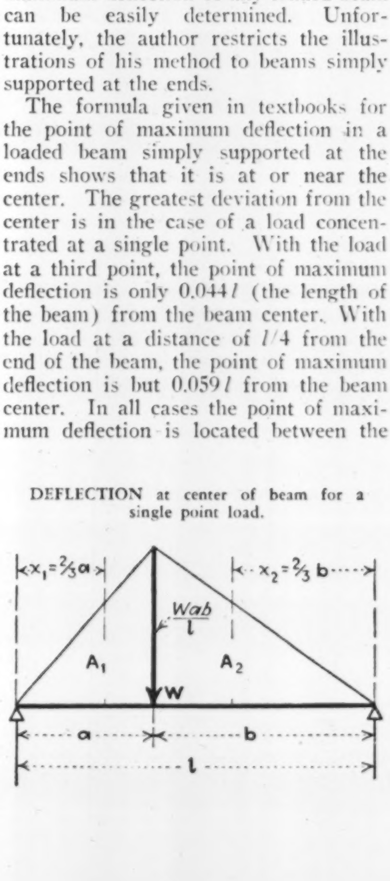
Letters to the Editor

Beam Deflection

Sir—Although the literature on the deflection of beams has become so extended that it may seem superfluous to write further on the subject, it seems worth while to call attention to a recent (1934) monograph of the Association of Engineering and Shipbuilding Draughtsmen (96 St. Georges Square, London), entitled "Some Notes on Deflection—Application of Area Moment Method to Uniform Section Beams," by W. R. Thomson. The author states that in these leisurely prepared notes he has endeavored to present a method, which he believes to be original, whereby the maximum deflection of any loaded beam can be easily determined. Unfortunately, the author restricts the illustrations of his method to beams simply supported at the ends.

The formula given in textbooks for the point of maximum deflection in a loaded beam simply supported at the ends shows that it is at or near the center. The greatest deviation from the center is in the case of a load concentrated at a single point. With the load at a third point, the point of maximum deflection is only $0.044l$ (the length of the beam) from the beam center. With the load at a distance of $l/4$ from the end of the beam, the point of maximum deflection is but $0.059l$ from the beam center. In all cases the point of maximum deflection is located between the point of loading and the beam center.

DEFLECTION at center of beam for a single point load.



Mr. Thomson first finds the deflection under the point load nearest the beam center. This is easily done, as the bending-moment diagram can be divided into triangles. He then by an ingenious method determines the increase of deflection from this point to the center. The sum of the two results is the desired deflection at the beam center. His general equation is:

$$\text{Deflection} = \frac{1}{lEI}\left\{ \begin{array}{l} bA_1x_1 + aA_2x_2 + \\ \dfrac{1}{2Ml}(A_2x_2 - A_1x_1)^2 \end{array} \right\}$$

in which a and b are the shorter (left end) and longer (right end) divisions of the beam length l; A_1 and A_2 are the areas of the moment diagrams on the left and right sides respectively of the point load W, with x_1 and x_2 the distances of their centers of gravity from the ends of the beam. The bending moment at the loaded point is denoted by M. The equation for a single point load (as shown on the sketch) reduces to $\frac{Wa}{3lEI}\ \frac{b(l^2 + 2ab)}{6}$. Comparing this with the formula for actual maximum deflection, $\frac{Wa}{3lEI}\left\{\frac{1}{3}b(l + a)\right\}^{3/2}$, the ratio (when $a = l/4$) of the approximate to the correct deflection is 112 to 110, an error of less than 2 per cent. Mr. Thomson considers $a = l/4$ the extreme deviation. For a load placed at the third point the error is less than 1 per cent.

In sketching the bending-moment diagram for a system of concentrated loads, convenient triangles are automatically formed by a unique and simple diagram. In drawing the bending-moment diagram for distributed loads, use is made of the bounding *triangle* to the parabola.

The method outlined by Mr. Thomson has its limitations. He rather overestimates the time saved by its use. Still, the monograph is well worthy of study throughout.

ROBINS FLEMING.

New York City,
April 11, 1935.

ENGINEERING NEWS-RECORD

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May 23, 1935

Vigilance Needed

IF ENGINEERS are to keep their rights from being encroached upon, they must maintain a close watch upon the many bills introduced into federal and state legislatures. The need for such watchfulness was stressed by Arnold G. Chapman, retiring president of the New York State Society of Professional Engineers, in reviewing the legislative work of that society at its annual meeting. Occasionally legislation is proposed with the evident purpose of limiting the activities of engineers, but more frequently the limitation is hidden in some innocent-looking phrase. A case in point is found in proposed changes in the architects' practice act in Pennsylvania, now before the legislature, noted in our news pages this week. Generally, the proposed changes are desirable and in conformity with current efforts to limit the use of the terms "architect" and "engineer" to those qualified to use them; but one apparently minor change in the exemption clause would practically exclude engineers from all building work in Pennsylvania. Similarly the activities of engineers in appraisal work or as expert witnesses in legal cases may be jeopardized by laws that would appear to have no bearing upon the profession. This condition calls for eternal vigilance supplemented by the ability to recognize the many ways in which the work of engineers might be limited. The work involved is arduous and time-consuming, but it provides a most fruitful field of service for the professional societies in the interests of their membership.

Industry Too?

MUST INDUSTRY as well as men go on relief to get a job from the works program? Last week's Washington report suggesting that such a thought was being entertained is about as discouraging as anything that has yet come to pass in connection with the program—as discouraging as the whole history of administrative procrastination and political delay that is wasting the best part of the current season. If the thought of limiting purchases to industries that rely on relief labor is really put into effect, it will bring more delay and further reduce efficiency; but most of all it will operate to the destruction of industrial morale just as the pauper's oath requirement for jobs has lowered individual morale. It is difficult to square the plan with the commendable action of the FERA in shutting down Ohio relief-worker manufacturing operations because of illegitimate competition with existing industry. That action revealed an understanding of the vital need for allowing normal industry to recover without interference by relief operations. The new plan, however, suggests a total absence of such understanding. Nothing now in sight is more likely to paralyze the relief-works program, to throttle and depress the normal enterprise of industry and business, than an attempt to allocate program purchases on the basis of relief-roll men employed in the factories. Let it be hoped that the report will prove to have been empty rumor, or, if it was well-grounded, then at least that saner counsel will yet prevail!

Fifty Years

FIFTY YEARS AGO engineering and industrial technology were entering an era of far-reaching transformation. An associated development saw its beginning at that time, and it is one that invites special notice on the occasion of James H. McGraw's fiftieth anniversary of technical publishing. The current chronicles of technology, the technical and industrial journals, began their growth to modern stature with the acceleration of industrial change that marked the eighties, and in the half century following they advanced hand-in-hand with the arts themselves. What they accomplished may be roughly appraised by contemplating the probable course of industrial and technical evolution had the journals not existed. In a word, it is not extravagant to say that they helped to bring about the vast improvement of the world's working equipment by which civilized life as we know it was made possible. And no small part of their contribution was leadership in developing the motive thought, the philosophy of industrial activity, which in turn exerted profound influence on economic and social thinking as a whole. For their part in making possible this service to the world, the creative pioneers who guided the growth of technical journalism are entitled to grateful appreciation.

Passamaquoddy Power

SINCE the Passamaquoddy power project was first put before the PWA for financing about a year ago, details of the proposed project have been kept from the public. Even now, after an allotment of $10,000,000 to start work on an undertaking estimated to cost over $36,000,000 has been recommended to the President, the only drawing that is available is a map showing the location of the proposed dams and power houses. Two inferences might be drawn from this secretiveness concerning the engineering details of the proposed structures, either that they have not been worked out in detail or else that they show all too clearly that the project cannot be built for anything like the estimated cost. If the former is the case, the statement that the work can start in five days is ridiculous; if the latter, the country is being deliberately deceived into putting vast sums into a scheme whose economic justification is questionable. How to harness the vast power of the tides has been a fascinating subject for speculation for many years, probably for centuries. That, however, is no reason why this country should be rushed into spending over $36,000,000 on a project concerning which so little is known. If $10,000,000 must be spent in Maine this year to combat unemployment, it can be better spent on roads, streets, schools and other current needs while the engineering and economic problems of the Passamaquoddy power project are studied dispassionately.

One More River

TO CALL TO MIND an incident that was mentioned in this column on at least one prior occasion, a survey of the prospects of bridge construction made some fifteen years ago led certain bridge engineers of much experience to conclude that the principal stream crossings necessary to accommodate traffic had already been built, and that therefore the prospects of large bridge construction were not bright. The error of this forecast is now well established. Beginning with the Bear Mountain Bridge, a long series of great bridging enterprises marked the record of construction so prominently as to rank the past decade as the leading bridge-building period of all history. And with each completed undertaking it is found that there is yet one more river to cross. The Omaha bridge represents the latest completed work, but neither it nor the New Orleans bridge nor the two great San Francisco structures will satisfy all the requirements of our steadily increasing demand for easier and more frequent road connection over major stream barriers. Nor do bridges when once built remain adequate for very long, as illustrated most clearly by the Brooklyn Bridge, whose projected modernization we discussed some weeks ago. The number of truly modern structures, able to carry traffic safely, conveniently and comfortably, is after all quite limited—in a sense Camden is the first of them in point of time, though it is barely ten years old. Thus a vast amount of rebuilding of great bridges will have to be done in the next generation, in addition to the construction of new river crossings for adequate service of transportation.

Roads for Speed

HIGHWAY DESIGN for speeds of 90 and 100 miles an hour is advocated in a strong statement by R. H. Baldock in another part of this issue. As state highway engineer of Oregon, Mr. Baldock is putting his belief into practice in new main road design. This action commands credit for its courageousness. It is not easy for a public highway official to take a course diverging so sharply from the direction of past and current belief in speed restriction. All law and ordinance regulating highway traffic looks upon speed as a hazard in itself, and most commentators on the present partial and undigested traffic-accident records support this conclusion. The challenge of the Oregon highway officials to such tradition is bold and timely.

Highway engineering is entering an era of express-road design and construction which is for the moment obscured by depression conditions that have centered attention on secondary roads and the wide distribution of relief-work road money to spread needed employment. The secondary and farm-to-market roads have rightful place in road-system development, but their improvement cannot replace the service to traffic of the main road. And traffic volume and speed are increasing at a rate that makes old standards of main-road design obsolete. The builder of these roads has to reorganize his thinking to new requirements of travel speeds and volumes not only as they exist at present but as they are certain to exist in another ten or twenty years.

The design of highways for greater speeds is a fundamental requirement of highway progress. No artificial barrier of law or accident propaganda will succeed in holding back increased highway travel speeds. They are an economic force of road transportation that cannot be resisted but must be provided for in road design. It does not mean revolution to make such design provisions. Correction of a few design elements is the only necessary beginning, as the current Oregon practice described by Mr. Baldock clearly indicates. Increase in the width of surfaced roadway and in sight distances at curves, at intersections and side-road entrances, and intelligent alignment and curve design are the vital requirements; they are all within the engineering and financial reach of the main-road builder if he will boldly grasp them.

The new era of main and express-road design and construction offers a brilliant future for highway engineering. Design and construction for much greater travel speeds is an inevitable requirement of the roads of this new era. The requirement can be met if the engineer will boldly cast aside legal speed limits as objectives of design. By doing this he risks no violation of present speed restrictions, and he has prepared for a future when these restrictions will be obsolete.

Wasting Time

SIX WEEKS have passed since the President signed the relief-works bill. To a nation which had been led to expect prompt action in putting money and men to work, the results are little short of heartbreaking. Principal accomplishment has been the setting up of an organization which seems ill-fitted to reduce red tape and impart immediate impetus to the program.

Even the announcement last week that the Advisory Committee on Allotments had recommended a billion dollars worth of projects to the President hardly rates as an accomplishment since most of the items were earmarked in the bill itself. The President has approved none of the allotments, and procrastination in dealing with the fundamental problem of wage rates raised a further obstacle to bid-taking. Debate is still in progress on terms of non-federal grants and loans, so that city officials must mark time in their planning. The entire preparatory part of the 1935 season has already passed, and even the fastest work, road construction, has not yet been allowed to issue a single call for bids. Critical time is being wasted, and with it that very necessary ingredient, public enthusiasm, is waning.

Another feature of the situation is the type of activity which too many local officials have embraced. Chafing under Washington delays, they have fallen to wasting time in promoting projects which a little reflection would reveal are entirely impracticable for the purposes of the moment. In Cleveland a committee has been appointed to resurrect the subway plan. If $40,000,000 of federal funds can be obtained, they say, the subway can be built. Nowhere is there any authority or precedent for the federal government to finance such a municipal project. Besides, relief works money must be spent in two years; subways require at least 5 years to build. Towns along the Ohio River look longingly for a federal project to remove pollution from the river, presumably by wholesale building of sewage-treatment plants. In New York, the tremendous Sixth Avenue subway is receiving mayoral ballyhoo, but it can hardly have a chance considering the time limit on the federal funds. Such projects are enticing; they make good newspaper copy; they get their sponsors into the headlines. But they waste valuable energy that could be used to better advantage on less spectacular but more practical planning.

A little less day-dreaming on the part of local officials, a little more speed on Washington's part in announcing the rules that will apply to non-federal projects, would aid progress in the relief-works program.

CURRENT NEWS

Army Engineers Get $102,186,500 For River Work

THE Advisory Committee on Allotments, on May 16, recommended to the President that $102,186,500 of the relief works fund be turned over to the Corps of Engineers for river and harbor work. The following are the principal items covered by the proposed allotment:

Major undertakings

Mississippi River between the mouth of the Missouri and Minneapolis—$25,000,000 to continue the work on the 9-ft. slackwater channel. A total of $37,000,000 is required to complete the work.

Missouri River (Kansas City to Sioux City)—$10,000,000 for the construction of dikes and revetments.

Fort Peck Dam—$16,000,000 for the continuation of that work. Previous allotments and appropriations amount to $50,000,000 and the total estimated cost is $86,000,000.

Passamaquoddy Bay, Eastport, Maine—$10,000,000 for start of work on a tidal power project. (See separate news item for details.)

Tygart River Dam, Grafton, W. Va.—$2,000,000 for the continuation of construction of the flood control reservoir.

Kanawha River, West Virginia and Ohio—$5,580,000 to continue modernization of the navigation facilities.

Chesapeake and Delaware Canal—$5,107,000 for dredging and bank revetment.

Beaver and Mahoning Rivers—$5,000,000 for dredging and the construction of locks, dams and reservoirs to improve the navigation facilities of the river. Total estimated cost $37,000,000.

Cape Cod Canal—$5,000,000 for continuation of the dredging work in the canal and in the approach channels.

New York State Barge Canal—$5,000,000 for the enlargement of the canal between Oswego and the Hudson River, including dredging and bridge alterations. Total estimated cost $27,000,000.

Other river work

Lesser amounts were set aside for river and harbor work as follows: Dredging in the Delaware River between Philadelphia and Trenton, $2,900,000; $2,800,000 for dredging, lock and dam construction in the Allegheny River to East Brady, Pa.; $2,671,000 for dredging work in connection with the Lake Okeechobee work in Florida; $2,000,000 for channel deepening in New York Harbor; $1,800,000 for dredging in San Diego Harbor; $1,760,000 for channel dredging at Buffalo; $1,684,000 for river work in the Missouri below Kansas City; $1,000,000 for breakwater construction in the outer harbor at Los Angeles; $1,463,000 for dredging in Tampa, Fla.; $1,200,000 for raising Emsworth Dam on the Ohio, and $1,000,000 for river work in the Mississippi from the mouth of the Missouri to Cairo

New River Power Case Dismissed by Supreme Court

The United States Supreme Court, on May 20, dismissed the suit brought by the federal government to require the owners of the Hawks Nest hydro-electric power plant on the New River in West Virginia to take out a federal license.

The federal government claims jurisdiction over the power plant on the ground that control of the flow of waters on the New River affect the navigability of the Kanawha River into which it flows.

"Control of navigation by the United States may be threatened by the imminent construction of the dam, but not by permission to construct it," said the court. For that reason and because no controversy between West Virginia and the United States within the range of judicial power is disclosed, the court dismissed the Hawks Nest case, involving the important question of jurisdiction over a tributary on which there is no actual navigation. Probabilities are that the government in its effort to prevent the construction of a dam without a license from the Federal Power Commission will have to bring its suit in the district court.

Allotments Approved for First Quarter of Relief Works Fund

ALLOTMENTS of $1,091,802,200 for non-federal projects, highway construction, grade crossing elimination, army engineer work, land resettlement, and housing were approved by the Advisory Committee on Allotments on May 16. Before these allotments become effective they must be approved by the President.

Highway work

Allotment of $500,000,000 to the Bureau of Public Roads was recommended, of which $200,000,000 is for highway, road and street construction, $200,000,000 for grade separation, and $100,000,000 for obligations incurred under the Hayden-Cartwright Act of 1934. (For details see the table given below.)

An allotment of $102,186,500 to the Corps of Engineers, including $10,000,000 for the Passamaquoddy power project in Maine, also was recommended. Details are given in a separate news item.

The committee recommended that the

DISTRIBUTION OF HIGHWAY AND GRADE-SEPARATION FUNDS UNDER ALLOTMENTS APPROVED MAY 16, 1935

State	Highways, Roads, and Streets	Highway-Railway Grade Separation and Protection	Total	Unappropriated Balance Under the Hayden-Cartwright Act
Alabama	$4,151,115	$4,034,617	$8,185,732	$2,129,921
Arizona	2,569,841	1,256,099	3,825,940	1,320,967
Arkansas	3,352,061	3,574,060	6,926,121	1,714,024
California	7,747,928	7,486,362	15,234,290	3,966,103
Colorado	3,395,262	2,631,567	6,026,830	1,743,003
Connecticut	1,418,709	1,712,684	3,131,393	727,434
Delaware	900,310	418,239	1,318,549	461,697
Florida	2,597,144	2,827,883	5,425,027	1,330,671
Georgia	4,988,967	4,895,949	9,884,916	2,556,745
Idaho	2,222,747	1,674,479	3,897,226	1,138,743
Illinois	8,694,009	10,307,184	19,001,193	4,460,700
Indiana	4,941,255	5,111,096	10,052,351	2,544,481
Iowa	4,991,664	5,600,679	10,592,343	2,559,180
Kansas	4,994,975	5,246,258	10,241,233	2,558,837
Kentucky	3,726,271	3,672,387	7,398,658	1,909,155
Louisiana	2,890,429	3,213,467	6,103,896	1,481,966
Maine	1,676,799	1,426,861	3,103,660	855,793
Maryland	1,750,738	2,061,751	3,812,489	905,029
Massachusetts	3,262,885	4,210,833	7,473,718	1,675,237
Michigan	6,301,414	6,765,197	13,066,611	3,226,284
Minnesota	5,277,145	5,395,441	10,672,586	2,712,775
Mississippi	3,457,552	3,241,475	6,699,027	1,770,113
Missouri	6,012,652	6,142,153	12,154,805	3,086,870
Montana	3,676,416	2,722,327	6,398,743	1,884,867
Nebraska	3,870,739	3,556,441	7,427,180	1,982,182
Nevada	2,243,074	887,260	3,130,334	1,151,178
New Hampshire	945,225	822,484	1,767,709	484,731
New Jersey	3,129,805	3,983,826	7,113,631	1,610,439
New Mexico	2,871,397	1,725,286	4,596,683	1,470,850
New York	11,046,377	13,577,189	24,623,566	5,663,960
North Carolina	4,720,173	4,823,958	9,544,131	2,420,470
North Dakota	2,867,245	3,207,473	6,074,718	1,469,483
Ohio	7,670,815	8,439,897	16,110,712	3,932,506
Oklahoma	4,580,670	5,004,711	9,585,381	2,342,590
Oregon	3,038,642	2,334,204	5,372,846	1,548,907
Pennsylvania	9,347,797	11,483,613	20,831,410	4,795,394
Rhode Island	989,208	699,691	1,688,899	507,286
South Carolina	2,702,012	3,059,956	5,761,968	1,385,477
South Dakota	2,976,454	3,249,086	6,225,540	1,523,821
Tennessee	4,192,460	3,903,979	8,096,439	2,151,495
Texas	11,989,350	10,855,982	22,845,332	6,145,626
Utah	2,067,154	1,230,763	3,297,917	1,066,345
Vermont	924,306	729,857	1,654,163	474,003
Virginia	3,652,667	3,774,287	7,426,954	1,882,693
Washington	3,026,161	3,095,041	6,121,202	1,553,206
West Virginia	2,231,412	2,677,937	4,909,349	1,140,167
Wisconsin	4,823,884	5,022,683	9,846,567	2,470,918
Wyoming	2,219,155	1,360,841	3,579,996	1,143,856
Dist. of Columbia	949,496	410,804	1,360,300	474,889
Hawaii	926,033	453,703	1,379,736	486,921
Eng. & Adm. Res.	5,000,000	4,000,000	9,000,000	....
Totals	$200,000,000	$200,000,000	$400,000,000	$100,000,000

federal government participate to the extent of $100,000,000 in a $205,714,000 public works program sponsored by the state of Wisconsin. The character of the work covered by the Wisconsin plan is given separately.

Housing

Allotment of $249,860,000 to the housing division of the Public Works Administration for 67 slum clearance and low-rent housing projects was approved. Thirty-six of the projects were among those affected by the impounding of $110,000,000 of the PWA housing division funds, and 31 projects are new applications. All allotments are tentative, the ultimate development of each depending upon the ability of the housing division of the PWA to assemble sites at a price which will permit the construction of true low-rent housing.

The resettlement administration was allotted $100,000,000 for its program.

The committee recommended an allotment of $7,500,000 toward the $25,245,000 Ward's Island sewage treatment plant at New York, primarily for the construction of intercepting sewers in Manhattan and the Bronx and tunnels to the treatment plant. It is recommended that the allotment be considered a 30 per cent grant subject to New York City financing the balance of the cost. About $5,500,000 already has been spent on the Ward's Island plant.

PWA projects

The committee recommended an allotment of $1,620,000 to the PWA for 19 non-federal projects, partly on a 70-30 loan and grant basis and partly on a 30 per cent grant basis.

An allotment of $446,500 to the Alaska Road Commission for an anchorage at Matanuska Road and for the construction of 30 miles of road in the newly colonized Matanuska Valley in Alaska was approved.

River work in Texas

An allotment of $20,000,000 for power, flood control and irrigation in Texas was recommended to be made to the Lower Colorado River Authority. Of this amount $5,000,000 will be transferred to the Bureau of Reclamation to aid in financing that portion of the project relating to flood control, and the remainder will be allotted to the PWA for a combined loan and grant to the Lower Colorado River Authority. When the agreement with the PWA has been completed, an existing PWA loan allotment of $4,500,000 for completion of the Hamilton Dam project in Bluffton, Texas, will be rescinded.

The balance of the money, about $200,000, has been set aside for administrative expenses.

Wagner Labor Bill Passed by Senate

The Wagner Labor Disputes Bill passed easily through the Senate on May 16 by a vote of 63 to 12. An amendment proposed by Senator Tydings to prevent coercion of workers by labor unions was overwhelmingly defeated.

A permanent labor relations board of three members would be set up under the Wagner bill with authority to supervise employee elections and to establish collective bargaining.

Government Undertakes to Build Passamaquoddy Power Project

THE federal government will take over the construction of the long-discussed Passamaquoddy Bay tidal power project in Maine, if the recommendation of the Advisory Committee on Allotments that $10,000,000 of the relief works funds be allocated to the Corps of Engineers to start construction of that project is approved by the President.

Project reduced in size

As originally proposed by Dexter P. Cooper several years ago, the project included all of Passamaquoddy Bay (mostly in the province of New Brunswick) Cobscook Bay in Maine, and several dams to shut off the two bays from the ocean and to separate them into an upper and lower pool. The present project, as shown in the accompanying map, includes only Cobscook Bay in the tidal power scheme and adds a pumped-storage reservoir to be located along the coast south of Cobscook Bay.

As has been the case ever since the project was first put forward, no engineering drawings of the proposed dams, control gates, and power houses have been made public. About a year ago the project was put up to the Public Works Administration for financing, but it was disapproved by the PWA board of review, and no details other than the estimated cost of $47,000,000 have been made public. The present project, according to the official statement, is designed to create and maintain a low level pool in Cobscook Bay, and to use the difference in head between this pool and the ocean, which will vary with the tide, to generate electric energy. The initial development will utilize the head above 5 ft. in a main power station with an installed capacity of 166,670 kva., but the powerhouse will be constructed for a maximum capacity of 366,874 kva., with 12 units not installed. The estimated cost is placed at $36,284,000. The normal range of tide in Cobscook Bay is 18 ft.

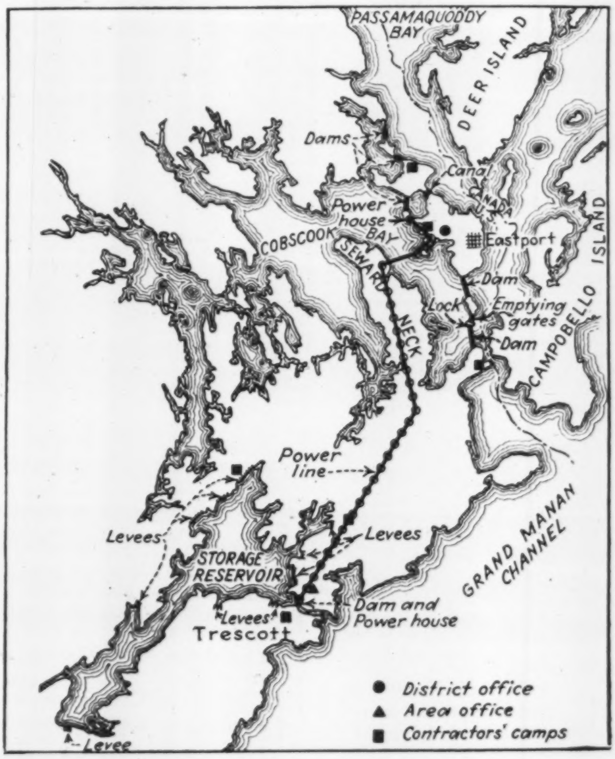
The storage reservoir will have a surface elevation of 130 ft. above sea level, and surplus power will be used to pump water into this reservoir, to be converted into electric energy during the low tide period.

Difficult work involved

The emptying gate which will control the level of the Cobscook pool, and the navigation lock, will require a cofferdam 75 ft. in depth, capable of withstanding a maximum of 20 ft. tidal range and ocean storms. Three of the rock dams are to be placed in water in excess of 100 ft. in depth and subject to high velocity due to the periodic reversing of the tide.

The Corps of Engineers has passed upon the practicability but not upon the economics of the project.

Major Philip B. Fleming, of the Corps of Engineers, who acted as deputy administrator of public works from September, 1934, until May, 1935, is to be engineer in charge of the Passamaquoddy project for the army engineers.



THE TIDAL PROJECT originally designed to include Passamaquoddy Bay now is confined to the much smaller Cobscook Bay supplemented by a pumped-storage reservoir.

Wage Rates Lower Than Those of the PWA Established for Relief Works Program

(Washington Correspondence)

WAGE SCALES at rates far below present industrial wages have been prescribed by the President for the relief works program. The wages range from $19 monthly for unskilled labor in the far South to $94 monthly for professional men in New York. The establishment of wage and hour schedules clears a big hurdle in path of the relief works program but work cannot start until the field organization of the Works Progress Administration is completed and conditions governing the amount of grants on PWA projects are fixed.

The rate was not built on a comparative basis but rather on a population and regional basis. Four regions, roughly consisting of the deep Southern, middle Southern, Central and Northern states, were set up. Basic rates in these regions differ and actual wages are further determined by the population of the largest city in the county where the work is being done.

Wages depend also upon the class of work done. In each of the four geographic sections the rates differ for types of work; unskilled, intermediate, skilled, and professional as follows:

city will in general apply to contiguous urban areas even though the urban areas are not in the same county. The President's order also states that wages throughout any county should be the same as in the largest municipality in the county, except if local conditions justify a different rate. In order to allow for any necessary adjustment the Works Progress Administrator may adjust rates in any locality by not more than 10 per cent.

The Progress Administrator will have the power to exempt projects, or portions of projects from the wages set down. The wage rates will be determined, subject to approval by the Works Progress Administration, and in accordance with local conditions, by the local agency having charge of the project.

Supervisory and administrative employees will be exempt and for those projects under state and local relief administrations, pending the transfer of workers to the new program, the present rules will be in force. In so far as is possible, workers will be taken from the relief rolls.

Except in the case of an emergency, and supervisory and administrative employees, persons working under the new wage schedule shall have the hours fixed by the Progress Administrator, Harry L. Hopkins, but the maximum will be an 8 hr.-day and a 40 hr.-week. The 30-hr. week will remain in effect on PWA projects, but double shifts will be used to get work done on time. This is also evidence of Mr. Ickes' effort to keep public works in the front line trenches in the relief works program, and to maintain emphasis on permanent construction as against projects of a "make-work" character. Mr. Ickes expects that PWA present wage schedules of 40, 45, and 50 cents for common labor and $1, $1.10 and $1.20 for skilled labor regionally, will be maintained, although the President's executive order provides that rates on such non-federal projects shall be determined by the municipality initiating the project, in accordance with local wage conditions, subject to approval of the PWA state director.

SCHEDULE OF MONTHLY EARNINGS

COUNTIES IN WHICH THE 1930 POPULATION OF THE LARGEST MUNICIPALITY WAS

Regions*	Over 100,000	50,000-100,000	25,000-50,000	5,000-25,000	Under 5,000
Unskilled Work					
REGION I	$55	$52	$48	$44	$40
II	45	42	40	35	32
III	35	33	29	24	21
IV	30	27	25	22	19
Intermediate Work					
REGION I	65	60	55	50	45
II	58	54	50	44	38
III	52	48	43	36	30
IV	49	43	38	32	27
Skilled Work					
REGION I	85	75	70	63	55
II	72	66	69	52	44
III	68	62	56	48	38
IV	68	58	50	42	35
Professional and Technical Work					
REGION I	94	83	77	69	61
II	79	73	66	57	48
III	75	68	62	53	42
IV	75	64	55	46	39

*Regions include the following States:

I. Connecticut, Maine, Massachusetts, New Hampshire, New Jersey, New York, Pennsylvania, Rhode Island, Vermont; Illinois, Indiana, Michigan, Minnesota, Ohio, Wisconsin; Arizona, California, Colorado, Idaho, Montana, Nevada, New Mexico, Oregon, Utah, Washington, Wyoming.

II. Iowa, Kansas, Missouri, Nebraska, North Dakota, South Dakota; Delaware, District of Columbia, Maryland, West Virginia.

III. Arkansas, Kentucky, Louisiana, Oklahoma, Texas, Virginia.

IV. Alabama, Florida, Georgia, Mississippi, North Carolina, South Carolina, Tennessee.

According to the order issued by President Roosevelt all wages will be on a monthly basis so that the worker will be paid for loss of time caused by weather or other conditions for which he is not responsible.

Some classes of work exempt

Exempted from the wage scale are the CCC, for which existing rules will remain in force, and the PWA for which wage rates will be determined by the state directors. Highway and grade-crossing elimination work will have wages fixed by the state highway departments. For federal building construction rates of pay are to be fixed by Congress.

Monthly wages that are applicable in a

Hours on permanent government buildings are governed by the provisions of the act of March 3, 1931.

Mr. Ickes is preparing recommendations governing the amount of grants on public works for which municipalities borrow the remainder. This will be liberal, but in the meantime applications are being considered on a 30-70 basis. PWA state engineers will serve as acting state directors in the decentralization of the PWA organization. Federal projects for which allotments have been made will get under way as soon as relief labor is supplied by the Hopkins organization.

Government Assists Wisconsin In State Public Works Plan

The Advisory Committee on Allotments has recommended to the President that $100,000,000 of the relief works fund be turned over to the state of Wisconsin to assist the state in its proposed state-wide public works program which is expected to cost $205,714,000. The allotment would be made conditional upon the state raising the remainder of the necessary funds, arranging to repay the federal government $30,000,000 of the $100,000,000, and upon the state's enacting satisfactory enabling legislation. It is contemplated that the projects proposed would take care of all the relief problems of the state.

The state program calls for $40,000,000 for urban low-cost housing; $25,000,000 evenly divided between state trunk roads and town roads; $15,000,000 for 67 CCC camps; $4,000,000 for a resettlement of 1,500 families now on sub-marginal land, and $10,000,000 for state buildings.

Other items in the program are: conservation of natural resources, $4,000,000; land liming, $3,500,000; soil erosion control, $3,000,000; marketing educational service, $1,000,000; rural electrification, $5,000,000; rural housing, $10,000,000; state planning board, $5,675,000, and administration, $4,000,000.

Stronger Code Enforcement Asked by AGC Boards

The Associated General Contractors of America, through its governing board and advisory board, which met recently in Washington, has issued the following statement:

"We feel the government has not fulfilled its obligations of supporting and enforcing the work of code authorities;

"Therefore, be it resolved that . . . the appropriate agencies of the government and the industry be advised that this association voices its dissatisfaction with past and present enforcement measures and stresses the importance of such adequate enforcement in the future in order to justify our further continued effort to make codes effective . . ."

The original report of this resolution in these pages made it appear that the AGC boards condemned the work of the visory board condemned the work of the code authority. The AGC boards wish to have it known that their expression of dissatisfaction was not to be interpreted as condemnation.

Parker Dam Project To Be Legalized by Johnson Bill

(Washington Correspondence)

FRAMED to legalize the status of all projects affected by the U. S. Supreme Court's decision in the Parker Dam case, April 29, the Johnson bill has been introduced in the Senate. There have been intimations that Arizona will not oppose this legislation if the Reclamation Service will undertake construction of the Parker-Gila irrigation project.

Tentative plans are understood to call for the allotment of $2,000,000 from the relief works appropriation to initiate the $20,500,000 project to irrigate lands in the Gila valley by water released from Boulder Dam to the Imperial diversion dam that probably will be contracted for soon.

The first segment of the Gila project includes 160,000 acres in the Yuma district east of the Yuma-Mesa project, but it is reported that, as the price of its assent to the Johnson bill, Arizona wants to include Willton Hills and the Mohawk Valley, adding at least 100,000 acres to the area to be developed.

Imperial Dam on doubtful list

Imperial Dam is part of the $38,500,000 All-American Canal project which was undertaken with a PWA allotment of $9,000,000 and will be carried on this year by an allotment of perhaps $20,000,000 from the relief works fund. Imperial Dam is one of the projects thrust into the doubtful category by the Supreme Court decision, but, by inference at least, was authorized by Congress in the Boulder Canyon Project Act of 1928, Section 10 of which reads, in part: ". . . the Secretary of the Interior is authorized to enter into contract or contracts with the said district [Imperial Irrigation district] or other districts, persons, or agencies for the construction, in accordance with this act, of said canal and appurtenant structures, and also for the operation and maintenance thereof, with the consent of the other users."

The Johnson bill was drafted for the specific purpose of authorizing all projects and validating all contracts for which allotments have been made by PWA under Title II of NIRA in anticipation that the court might hold, as it did with respect to Parker Dam, that the streams on which they are located are navigable.

Many projects authorized

Section 1 of the bill refers specifically to Parker Dam and Grand Coulee Dam, but in section 2 are included all projects planned or undertaken under NIRA "for the purpose of controlling floods, improving navigation, regulating the flow of the streams of the United States, providing for storage and for the delivery of the stored water, for the reclamation of public lands and Indian reservations and other beneficial uses and for the generation of electric energy as a means of financially aiding and assisting such undertakings."

In addition to authorizing such projects and validating all contracts heretofore executed in connection with them, the President is authorized to construct and maintain dams.

C. F. Loweth, Chief Engineer Of Milwaukee R.R., Dies

Charles F. Loweth, who until his retirement on May 1 had been chief engineer of the C. M. St. P. & P. R.R., died in Chicago, May 15.

Born in 1857 at Cleveland, Ohio, Mr. Loweth received his education at Oberlin College. In 1876 he began his engineering experience as a rodman with the Cleveland, Lorain, & Wheeling Ry. He was later employed as draftsman for the Santa Fe R.R. In 1881-1882 he served as engineer in the construction of the Des Moines & Northern and St. Louis, Des Moines & Northwestern railroads. Mr. Loweth was then employed as engineer with Raymond & Campbell, bridge builders, Council Bluffs, Iowa.

From 1883 to 1901 he was in business as a consulting engineer in St. Paul. During this time he designed and supervised construction of water works, sewerage and electric lighting systems for many cities in the Northwest. At the same time he was consulting bridge engineer for the Northern Pacific and other railways. Mr. Loweth then entered the service of the Milwaukee road as engineer and superintendent of bridges and buildings. In 1910 he became chief engineer and held that position until his retirement.

Mr. Loweth has served as president of the American Society of Civil Engineers and the Western Society of Engineers. In 1915 he was granted an honorary degree of civil engineer by the University of Wisconsin and in 1926 received an honorary doctor of engineering degree from Rose Polytechnic Institute.

American Assn. of Engineers Celebrates Twentieth Anniversary

A hundred founders and early members of the American Association of Engineers met in Chicago on May 15 to celebrate the twentieth anniversary of the founding of A.A.E. in that city. A. H. Krom, R. H. Holmes, W. J. Mauer and T. A. Evans, four of the first five original members, took part in the program, which was replete with reminiscences of early day efforts. Present-day memberships bear numbers upward of 44,000 and the Association today has 6,200 members.

While the meeting was not intended for the transaction of society business, a move was made toward formation of a new Chicago chapter. A welfare program for the next 20 years embodying the following points was proposed as a goal: (1) classification of all engineering positions, with specifications for every position and a minimum salary schedule; (2) a national employment service; (3) formulation of specific principles of good professional conduct to be made official by all engineering organizations, covering engineers in private practice and employed men equally; (4) uniform licensing laws with enforcement; (5) recommendations on engineering training, including emphasis on vocational guidance and incorporation of business courses in curriculums; (6) comprehensive legal service; (7) publicity and a more aggressive entry into civic affairs; (8) creation of a welfare foundation to cover unemployment insurance, accident insurance, and annuities and pensions.

Construction Code Amendments Receive NRA Approval

Approval of several amendments of Chapter I of the Construction Code has been announced by the NRA. The first of these increases the responsibility of the code authority with respect to amendments of divisional chapters. Before approval of such chapter amendments by NRA, they must be submitted to the Construction Code Authority for its consideration and examination as to the coordination of the proposals with various code provisions. This is similar to the function of the authority in regard to adding new chapters to the code.

An amendment to Sect. 1, Art. III, of Chapter I, provides that joint area agreements may be made between employers and employees in two or more divisions or subdivisions of the industry by mutual consent. Previously the code provided that area agreements could be for only one division or sub-division of the industry. In a number of cases it was seen to be an advantage to make agreements for two related divisions such as the plumbing and heating and piping divisions. It will also aid area agreement procedure in the South and in other sections where the special contracting divisions are not clearly differentiated.

Other amendments to Art. III of Chapter I provide for an impartial chairman for regional construction planning and adjustment boards, and remove the fixed number of representatives of employers and employees on area agreement boards.

Pennsylvania Legislative Bill Would Limit Engineers' Work

The filing of building plans and the supervision of building construction by engineers in the state of Pennsylvania would be prohibited to all practical purposes by a bill to amend the architects' practice act of 1919, introduced into the House of Representatives late in March and now before the House committee on education.

The bill amends the architects' practice act of 1919. Ostensibly its purpose is to strengthen the law as regards the use of the term architect and to limit the performance of architectural services. The major changes are not objected to by engineers; they are in line with similar changes made in engineer license laws. However, a proposed change in the exemption clause of the existing law has a direct bearing. The clause in question is as follows, the new words being in italic:

"This act shall not be construed to prevent persons other than architects from filing applications for building permits or obtaining such permits, nor shall it be construed to prevent such persons from designing buildings and supervising their construction provided *that the public safety or health is not involved and provided further that* their drawings are signed by the authors with their true appellation as engineer, or contractor, or builder, or carpenter, but without the use in any form of the title architect." As public health or safety is involved in any building of any magnitude, this section would appear to limit the preparation and filing of plans and the supervision of construction of all but very small structures to architects.

Washington Highlights

By Paul Wooton

Washington Correspondent
Engineering News-Record

Washington, May 21—Despite their various setbacks, the TVA amendments will go through in much the form desired by the Administration.

There is no expectation that Congress will approve government ownership of the Federal Reserve banks. Official proposals to that effect are seen as a move to checkmate opposition to the banking bill in its present form.

Prospects for the Wagner labor disputes bill and the Guffey coal bill depend upon developments with respect to NRA. Failure to extend NRA for two years will greatly improve the chances of these measures.

Some regard the forthcoming decision in the Schechter case as more important than in the gold cases. It is expected to determine how far cooperation may go in an effort to avoid undesirable features of unbridled competition.

The RFC is meeting arguments that loans for new machinery decrease employment by saying employment would suffer more in the long run if inefficient equipment is used. With the profit motive appearing, applications for industrial loans have increased. At first these loans were sought chiefly for the purpose of keeping businesses going and to hold personnel organizations together.

Kettering Eulogizes Research at Joint Activities Meeting

An enthusiastic eulogy of research as the prime agency of industrial revival and progress by Charles H. Kettering, Director of Research of General Motors Corporation, and an equally enthusiastic advocacy of cooperation in the E.C.P.D. movement as the means of raising the status of the engineering profession, by Clarence F. Hirshfeld, Director of Research of the Detroit Edison Company, were prominent features of the dinner meeting on joint activities of engineering societies held at New York on May 20. About one hundred leading members of the principal national engineering societies attended.

Mr. Kettering decried the pessimistic view that the development of the world is largely completed, and said that we have not yet begun to build a world as it should be. Research must be the leader to improvement. With respect to results of engineering research hitherto, great as they are, he likened them to random prospecting for oil; up to now we have only found driblets, he suggested, but some day we are going to strike a pool, and then the world will be sure to undergo rapid change.

Dr. Hirshfeld stated the case for the E.C.P.D. with accustomed idealism. The growth of the engineer himself to larger capacity and performance and the growth of his recognition by the community, are the prime objectives of the movement. He emphasized particularly that it is in no way in competition or at odds with registration, but aims to support and supplement it.

Revised Cement Code Omits Cost Protection

Approval of a revised code for the cement manufacturing industry was announced by the National Industrial Recovery Board on May 13, following months of discussion of the code between the industry and the NRA. A clause of the code involving classification of sales was early suspended by the NRA, and in the following discussions numerous other changes were proposed. The present revised code is the outcome.

Leading among the changes made is the elimination of provisions for production control and for cost protection. The latter, a prohibition of selling below cost, was originally drafted by the NRA, but proved ineffective in practice. Production control provisions also were found not of value. A clause permitting arrangements for allocation of output under certain circumstances was also deleted in the revision.

Labor clauses were changed to substitute the straight 40-hour week (or 8 hours per day or 6 days per week) with time and one-half for overtime in place of a maximum 42-hour week averaged to 36 hours over a 6 months period. Minimum wage rates were not changed.

Price filing, with a 5-day waiting period, remains unchanged except that prices are hereafter to be filed with a confidential agent instead of with the code authority.

A section permitting members of the industry to file destination costs based on land grant or other special freight rates granted the federal government, or to meet such destination costs filed by others, was stayed, pending further study.

Scioto-Sandusky Project To Be Reconsidered

Following a recent report that the proposed flood-control project for the Scioto and Sandusky Rivers in Ohio was economically unwarranted, army engineers have agreed to review the conservation program there. The data used in the first report are said to have been obtained some time ago in connection with a proposed canal from Portsmouth on the Ohio River to Sandusky on Lake Erie. Additional and more recent information will be sought by the army engineers.

Plans for the project called for a ten-mile canal near Marion through which flood waters from the Scioto would be diverted to the Sandusky, and for the construction of reservoir.

Advance Public Works Planning Urged by New York Engineers

Setting aside of $50,000,000 of the $4,000,000,000 relief works fund and the allocation of this money to federal, state and municipal departments for advance planning of the work to be carried out under the 1935 relief act was urged upon the federal government by the New York State Society of Professional Engineers at its annual meeting held in New York City on May 18. The society, in its resolution, pointed out that for lack of funds state and municipal governments have dropped engineers and architects needed for planning the works program.

Contract work favored

The society went on record as favoring the abandonment of the present relief works methods and the substitution of contract work on all projects involving an expenditure in excess of $10,000.

Other resolutions adopted by the society called upon the state TERA to stop disregarding the engineer license law, and to adopt proper classifications for professional men, paying them more adequate compensation. The society also went on record as endorsing the report of the Metropolitan Committee on Engineers' Employment and Salaries and urged that the rates given therein be made effective.

Traffic accident studies

The society recommended to the national government that funds from the relief-works act be set aside for a national study to determine the causes of the major highway traffic accidents and to recommend methods for eliminating them.

In reviewing the activities of the society for the past year, Arnold G. Chapman, retiring president, pointed out that further changes in the laws affecting the practice of engineering and land surveying are needed, especially a change in the real-property law as it relates to property surveys, further tightening of the exemptions in the engineer license law and improvements in the civil service act. Mr. Chapman also urged that the society develop a standard form of contract for engineering services to builders.

Mr. Chapman pointed out the need for a constant review of bills submitted to the legislature in order to protect the interests of engineers.

Public works department

At the luncheon meeting, Willard S. Conlon, secretary of the National Society of Professional Engineers, reported upon the activities of that society in relation to the two bills now before Congress providing for the creation of the "Department of Conservation and Works" in place of the present Department of the Interior. Mr. Conlon reported that he had urged that the department be known as "Department of Conservation and Public Works."

New officers

The following officers were elected, John C. Riedel, president; vice-presidents, Alfred E. Roche and Fred H. Zurmuhlen; financial secretary, Ernest F. Fox; treasurer, Patrick M. Corry; directors, John Avery, Charles W. Burke, Joseph F. Lamb, Elmer C. Lawton, John L. Scanlon, Ole Singstad, Edward E. Stickney, Walter H. Weiskopf, Linford S. Stiles and James A. Pietsch.

The society now has an active membership of 1,600 and inactive membership of 500.

Twenty State Administrators Named For Works Program

Local projects which constitute the backbone of the $4,000,000,000 relief works program are waiting on the organization of state offices. As soon as Harry Hopkins, Works Progress Administrator, appoints his state deputies, projects can be submitted and the sifting process can begin even while the administration in Washington still is mulling over major questions of policy that will govern execution of the works program.

Mr. Hopkins is selecting the state works progress administrators personally and has named twenty, of which ten previously had directed relief activities in the state where they have been put in charge of the work program. Each state has likewise been divided into several employment areas, with local offices. The purpose of this decentralized organization, as explained by Mr. Hopkins, is to give everybody an opportunity to suggest projects and get them "in the works."

State administrators have been appointed as follows:

Alabama—Thad Holt, now director of the Alabama Emergency Relief Administration.

Delaware—Bankson T. Holcomb, of Harrington.

Florida—C. B. Treadway, now chairman of the Florida Emergency Relief Administration.

Georgia—Miss Gay B. Shepperson, now director of federal relief in Georgia.

Idaho—J. L. Hood, of Pocatello.

Indiana—Wayne Coy, now administrator of the governor's commission for unemployment relief.

Kentucky—George H. Goodman, now director of the Kentucky Relief Administration.

Louisiana—Frank H. Peterman, now director of federal relief in Louisiana.

Maryland—John N. Mackall, of Baltimore.

Michigan—Harry Lynn Pierson, of Detroit.

Mississippi—Wayne Alliston, of Jackson.

Missouri—Matthew S. Murray, of Kansas City.

New Mexico—Lea Rowland, of Roswell.

Ohio—Charles C. Stillman, also administrator of the FERA in Ohio.

Oregon—E. J. Griffiths, of Portland.

South Dakota—M. A. Kennedy, also administrator of the South Dakota emergency relief administration.

Texas—H. P. Drought, of San Antonio.

Virginia—William A. Smith, now administrator, Virginia emergency relief administration.

Washington—George H. Gannon, of Pullman.

Wyoming—Will G. Metz, also administrator of the state relief committee.

Bids Invited for Addition To Post Office at Vancouver

Bids have been invited by the Department of Public Works at Ottawa for the erection of the $1,000,000 addition to the post office at Vancouver.

The new section will be six stories high and have two basement levels. Construction is to be of steel and concrete frame with concrete exterior walls, granite facings and base of Haddington Island stone.

SOCIETY CALENDAR

AMERICAN INSTITUTE OF ARCHITECTS, annual convention, Milwaukee, Wis., May 28-30. THE PRODUCERS' COUNCIL will hold its 12th annual meeting in connection with the Institute meeting.

AMERICAN SOCIETY FOR TESTING MATERIALS, annual meeting, Detroit, June 24-28.

AMERICAN SOCIETY OF CIVIL ENGINEERS, annual convention, Los Angeles, July 3-5.

NEW YORK STATE SEWAGE WORKS ASSOCIATION, spring meeting, Poughkeepsie, N. Y., May 24-25.

PENNSYLVANIA SEWAGE WORKS ASSOCIATION, annual meeting, State College, Pa., June 24-26.

PROVIDENCE ENGINEERING SOCIETY at the annual meeting May 14 elected the following officers: Ellery L. Wilson, president; Clarke Freeman, vice-president; Alton C. Chick, vice-president; William R. Benford, secretary; E. L. Freeman, treasurer.

THE METROPOLITAN SECTION of the American Society of Civil Engineers at its annual meeting May 15 elected the following officers: president, V. T. Boughton; vice-president, E. M. Van Norden; treasurer, J. S. Langthorn; directors, R. E. Backenhus, C. W. Bryan, Jr., and William Heyman.

STATE EXAMINATIONS for registration of engineers and land surveyors are scheduled to be held in the near future as follows:

MICHIGAN—By the State Board of Examiners for Registration of Architects, Engineers, and Surveyors, June 13-15. For architects at the University of Michigan; for engineers and surveyors at the University of Michigan, Ironwood, Houghton, and Michigan State College.

Brief News

The TVA Engineers have started sinking test drills to determine foundations for the proposed Chickamauga dam on the Tennessee River, a few miles from Chattanooga. Other sites are still being considered by the Tennessee Valley Authority, but the results of the core borings now under way will determine where the $28,000,000 dam will be erected across the river.

Cost of Proposed Sewage Disposal Plant at Fields Point, Providence, R. I., will have to be increased from $475,000 to $605,000 because of an increase of 25 per cent in the price of materials. The total amount of bids received to date is $538,071, to which must be added a royalty fee of $30,000 to the maker of the disposal equipment. Changes in the design of the plant since the project was first studied in 1930 have reduced the cost over $100,000 and studies of operating costs have indicated a power cost reduction of $38,000 per year from the original estimate of $71,000.

Application of the Cairo, Ill., Bridge Commission for a permit to construct a toll bridge across the Ohio River at its mouth, is being opposed by the Mississippi Valley Association on the grounds that a bridge at that location would destroy the strategic advantages of the confluence of the Ohio and Mississippi rivers in time of war, and also that the bridge would interfere with navigation on the rivers. The association holds that no bridge should be built closer to the Mississippi River on the Ohio than the present railroad bridge, which is 3 miles from the mouth of the Ohio. The proposed bridge would cost about $2,000,000 and would be financed with PWA funds.

Construction of a Series of Large Dikes as a flood control project is being planned by East Hartford, Conn. The annual overflow of the Connecticut River has become a serious menace and has in the past resulted in costly damage to property. An allocation of $650,000 is being sought from the federal government to do this work. Federal engineers made a survey of the proposed project several months ago.

Personals

Chrystal Brown has been appointed superintendent of the Wanaque Reservoir by the North Jersey District Water Supply Commission.

Fred B. Crosby, for the past 17 years engineer of the Morgan Construction Co., Worcester, Mass., has been elected president of the Worcester Engineering Society.

George F. Cotterill, formerly Seattle port commissioner, has been retained by the city of Bellingham, Wash., as consulting engineer on public works projects.

John G. Clarke, former town engineer of Southbridge, Mass., has been appointed field engineer for the Worcester district on ERA projects.

J. M. Robertson, consulting engineer, has been appointed by the Montreal Board of Trade as its representative on the commission that is to study the need of a sewage disposal plant for the city.

John J. Crowe has been elected president of the American Welding Society. Mr. Crowe is engineer in charge of apparatus research for the Air Reduction Co., Jersey City, N. J.

Nathan L. Smith, who has been acting chairman of the Maryland state roads commission for the last few months, has become chief engineer, succeeding Harry D. Williar, Jr., who held the post for seven years.

Maj. Gen. Lytle Brown, former Chief of Engineers, U. S. Army, has been named commander of the Panama Canal department, to be effective upon the departure of Maj. Gen. Harold B. Fiske, the present commander, who is to retire in November.

John R. Peavy, former Mobile County engineer, has been appointed chief of the Alabama highway maintenance department. H. D. Burnum is serving as acting engineer in charge of the first division; R. D. Jordan has become head of the second division; J. B. Rutledge has charge of the third division; S. J. Cummings is in charge of the fourth division.

Major Roy D. Burdick, who for the past four years has been in charge of the procurement planning branch, office of assistant secretary of war, Washington, D. C., will go to Memphis June 15 as assistant to Col. Eugene Reybold, Memphis district engineer. Maj. Burdick will replace Lieut. Thomas A. Lane, assistant to the district engineer, who has been ordered to duty at Fort Belvoir, Va.

Obituary

HENRY E. MOORE, an engineer in the park department of New York City, died at his home recently after a short illness.

CHARLES E. ANGILLY, engineer in charge of structural design for the Los Angeles, Calif., Department of Water and Power, died there May 10 at the age of 46. Mr. Angilly, a member of the city department for nineteen years, was a member of the American Society of Civil Engineers.

CHARLES N. BLACK, 67, died at his home in San Francisco May 15. Prior to 1922 Mr. Black was manager of the New York office of Ford, Bacon & Davis and at the time of his death he was a director in that firm. He was born in New York City on March 16, 1867. He graduated from Princeton University with an A.B. in 1888 and an E. E. in 1890. In 1899 he entered the employ of Ford, Bacon & Davis.

E. E. PARKER, chief engineer, Wisconsin highway commission, died May 16 in Madison, Wis., at the age of 51. He was graduated from the University of Wisconsin in 1908 and for many years prior to his present position which he assumed two years ago had been city engineer of Madison. He was an active member of the American Society of Municipal Engineers and the Engineering Society of Wisconsin.

JAMES HUBERT VAN WAGENEN, commissioner of the International Boundary Commission, died in Washington, May 17, after a month's illness. Mr. Van Wagenen, who was 54 at his death, had for many years been prominent in Washington engineering circles. He had been at one time president of the District of Columbia section, American Society of Civil Engineers, and also of the Washington Society of Engineers.

AUGUSTUS W. JONES, 73, died in Washington, D. C., May 7. Mr. Jones graduated from Antioch College and the University of Ohio and later became chief engineer of several interurban railroads in Ohio. He also served as engineer of construction and location on the C.C.&O.R.R., and as chief engineer of the Montour R.R. Mr. Jones was at one time chief engineer of the John B. Carter Co. on railroad work in Maryland and Indiana. Since 1923 he has been sales engineer for the Kentucky Rock Asphalt Company.

CONSTRUCTION STATISTICS OF THE WEEK

ENGINEERING construction awards for the week total $34,299,000, a gain of $12,619,000 over last week. State and municipal awards amounting to $18,335,000 and federal work at $10,777,000 are responsible for this rise. Federal awards are the highest this year. Private awards at $5,187,000 are considerably below last week.

Gains are recorded in public buildings at $5,481,000; streets and roads at $6,096,000; waterworks at $2,045,000; sewers at $6,674,000; bridges at $903,000; and earthwork, irrigation, drainage and waterways at $7,409,000. Other classes of work are low.

The larger projects include: machinery for steel strip mill at Gary, Ind., for American Sheet and Tin Plate Co., $1,000,000; post office at San Antonio, Tex., $1,780,000; parcel post building, Detroit, Mich., $845,000; highway awards by Texas, $806,000; by Tennessee, $515,000; by Onondaga County, Syracuse, N. Y., $900,000; road surfacing materials for state highway department, Wisconsin, $1,112,000. At Eureka, Calif., contract was let for headworks and 20 mi. pipe line for Mad River Water Supply System, $1,124,000; at Chicago, Ill., for Divisions D, E, and F and Contract 3 for Southwest Sewage Treatment Works, $5,768,000. Dam 26 on the Mississippi River at Alton, Ill., $4,866,000 and levee work in West Atchafalaya Levee District, Louisiana, $1,385,000, were put under contract by the U. S. Engineer Office.

New capital includes $4,154,000 in state and municipal bond sales and $858,000 in PWA allotments for federal work. Reductions and rescissions of PWA allotments again exceeded reallotments with the net result of lowering the amount of PWA allotments outstanding by $4,513,000. The large allotments of relief works funds noted elsewhere in this issue have not been included as they have not yet been approved by the President.

CONTRACTS
(Thousands of Dollars)

	Weekly Average May 1934	Prev. 4 Weeks	Week May 23 1935
Federal Government	$3,144	$2,347	$10,777
State and municipal	14,597	10,901	18,335
Total public	$17,741	$13,248	$29,112
Total private	5,607	6,291	5,187
Week's total	$23,348	$19,539	$34,299

Cumulative to date:
1934...$574,432,000 1935...$516,063,000

NEW PRODUCTIVE CAPITAL
(Thousands of Dollars)

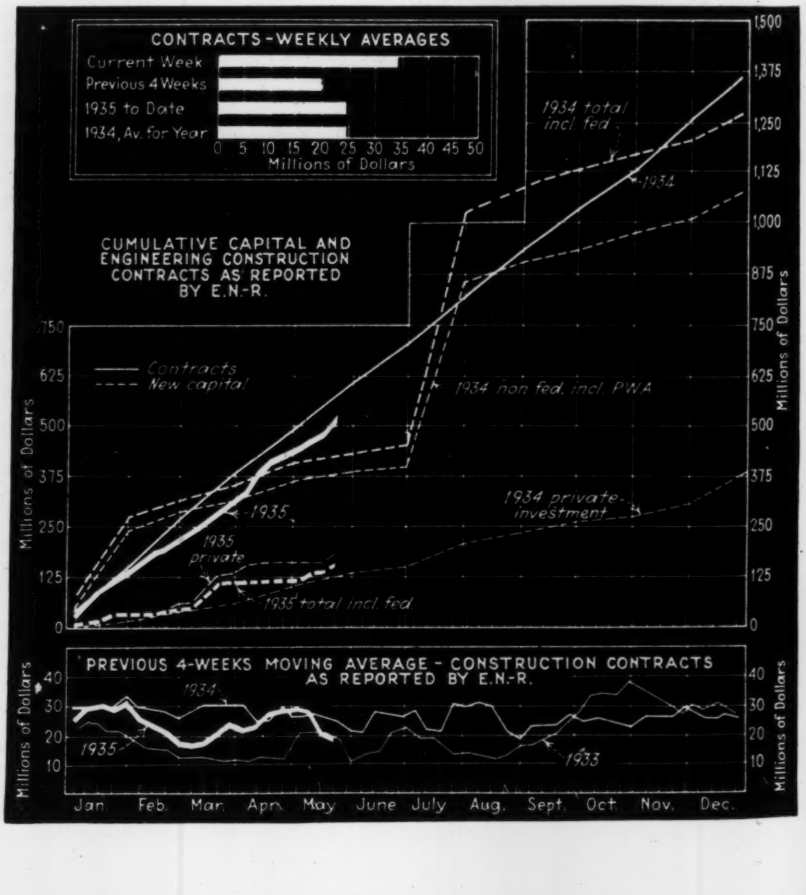
1935	Week May 23	Cumulative
State and municipal....	$4,154	$162,454
PWA allotments, S&M.	—4,513	—49,032
RFC loans, S&M......	....	84
Corporate issues	....	....
PWA allotments, private	....	8,619
Total, Non-Federal...	—$359	$122,125
PWA allotments, Federal Constr.	858	9,907
Total new capital....	$499	$132,032

Cumulative to date:
1934...$426,640,000 1935...$132,032,000

Cumulative to date:
1934...$416,874,000 1935...$131,533,000

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities, including the special highway funds; PWA private loans, and allotments for Federal construction. Negative amounts in PWA totals indicate excess of rescissions and reductions over reallotments and increases in outstanding allotments.

INDEX NUMBER

E.N.R.-Cost	1913 = 100	1926 = 100	E.N.R.-Volume	1913 = 100	1926 = 100
May, 1935	**194.06**	**93.28**	**Apr., 1935**	**130**	**57**
Apr., 1935	194.50	93.49	Mar., 1935	106	44
May, 1934	199.61	95.95	Apr., 1934	109	48
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50
1933 (Av.)	170.18	81.80	1933 (Av.)	102	47
1932 (Av.)	156.97	75.45	1932 (Av.)	127	56